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THE NORTHERN MAINE REGIONAL TREATMENT SYSTEM

James A. Barresi, et al

Northern Maine Regional Planning Commission

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April 1973

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Cost analyses of all viable options and alternatives were prepared, including capital and operating costs. Annual revenue requirements for each system were projected, including evaluation of current State and Federal grant-in-aid programs. Joint municipal-industrial treatment facilities proved the most economic course of action.

The technical studies of the research and development program were evaluated for water quality impact on the receiving waters, as determined by Companion River Basin Studies. (EPA)

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THE NORTHERN MAINE REGIONAL TREATMENT SYSTEM

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ABSTRACT

Detailed sampling, gaging and laboratory analyses determined current waste loads from the Aroostook-Prestile Basin's potato processing industry. Studies indicated that significant reductions in load could be accomplished by in-plant conservation. Biological treatment of the residual wastes, however, was found necessary.

Preliminary designs were prepared for numerous treatment and loading operations, including joint industry-municipal plants and regionally inter-connected systems. A transport-treatment channel system covering some eleven miles was shown to be technically feasible.

Cost analyses of all viable options and alternatives were prepared, including capital and operating costs. Annual revenue requirements for each system were projected, including evaluation of current State and Federal grant-in-aid programs. Joint municipal-industrial treatment facilities proved the most economic course of action.

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CONTENTS

Section		Page
I	Conclusions	1
II	Recommendations	5
III	Introduction	9
IV	Industrial Waste Studies	15
V	Treatment Plant Design Criteria	55
VI	Treatment-Transport System Analysis	73
VII	Regional Treatment Analyses	149
VIII	Capital and Annual Costs	217
IX	Cost and Ancillary Benefit Analyses	271
X	Discussion	281
XI	Acknowledgments	283
XII	Literature Review and References	285
XIII	Computer Design Program	297
XIV	Glossary, Abbreviations and Symbols	317
XV	Appendix	323
XVI	Summary Report	333

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FIGURES

PAGE

1 GEOGRAPHIC ORIENTATION PLAN 10

2 PROJECT CRITICAL PATH METHOD FLOW DIAGRAM 12

3 INDUSTRY LOCATION PLAN 16

4 SCHEMATIC PROCESS FLOW DIAGRAM - TATERSTATE-WASHBURN 18

5 SCHEMATIC PROCESS DIAGRAM - POTATO SERVICE INC 20

6 SCHEMATIC PROCESS DIAGRAM - CYR BROTHERS POTATO STARCH CO 23

7 SCHEMATIC PROCESS DIAGRAM - COLBY COOPERATIVE STARCH CO 26

8 SCHEMATIC PROCESS FLOW DIAGRAM - A & P PROCESSING PLANT 28

9 SCHEMATIC PROCESS FLOW DIAGRAM - MAINE SUGAR INDUSTRIES 32

10 PLOT - SURFACE LOADINGS VS BOD AND SS REMOVAL - EXISTING FACILITIES 52

11 COMMUNITY AND INDUSTRY LOCATION PLAN 74

12A & B INITIALLY PROPOSED TREATMENT-TRANSPORT CONFIGURATION 76 & 77

13A & B FINAL TREATMENT-TRANSPORT CONFIGURATION 80 & 81

14 PARABOLIC CHANNEL 83

15 WATER DEPTH IN CHANNEL VS INFLOW 86

16 WATER DEPTH VS INFLOW 87

17 VELOCITY VS INFLOW 88

18 VELOCITY VS INFLOW 89

19 TIME IN CHANNEL VS INFLOW 90

PAGE

20 CHANNEL SURFACE AREA VS INFLOW 92

21 CHANNEL SURFACE AREA VS INFLOW 93

22 DEPTH VS DIFFUSER SPACING 96

23 SCHEMATIC DIAGRAM - TREATMENT-TRANSPORT SYSTEM ALTERNATE 2 98

24 BOD CONCENTRATION VS CHANNEL DISTANCE - MINIMUM LOAD ALTERNATE 2 103

25 BOD CONCENTRATION VS CHANNEL DISTANCE - MEDIUM LOAD ALTERNATE 2 104

26 BOD CONCENTRATION VS CHANNEL DISTANCE - MAXIMUM LOAD ALTERNATE 2 105

27 MLSS CONCENTRATION VS CHANNEL DISTANCE - MINIMUM LOAD - ALTERNATE 2 106

28 MLSS CONCENTRATION VS CHANNEL DISTANCE - MEDIUM LOAD - ALTERNATE 2 107

29 MLSS CONCENTRATION VS CHANNEL DISTANCE - MAXIMUM LOAD - ALTERNATE 2 108

30 OXYGEN DEMAND VS CHANNEL DISTANCE - MINIMUM LOAD ALTERNATE 2 109

31 OXYGEN DEMAND VS CHANNEL DISTANCE - MEDIUM LOAD ALTERNATE 2 110

32 OXYGEN DEMAND VS CHANNEL DISTANCE - MAXIMUM LOAD ALTERNATE 2 111

33 SCHEMATIC DIAGRAM - TREATMENT-TRANSPORT SYSTEMS ALTERNATE 3 118

PAGE

34 TYPICAL SOLIDS CONCENTRATOR AND RETURN INSTALLATION, TREATMENT-TRANSPORT SYSTEMS 119

35 BOD/MLSS RATIO VS MLSS CONCENTRATION - MINIMUM CONDITIONS 121

36 BOD/MLSS RATIO VS MLSS CONCENTRATION - MINIMUM LOAD ALTERNATE 3 125

37 BOD/MLSS RATIO VS MLSS CONCENTRATION - MEDIUM LOAD ALTERNATE 3 126

38 BOD/MLSS RATIO VS MLSS CONCENTRATION - MAXIMUM LOAD ALTERNATE 3 127

39 MLSS CONCENTRATION VS DISTANCE - ALTERNATE 3 131

40 MLSS CONCENTRATION VS DISTANCE - ALTERNATE 3 132

41 MLSS CONCENTRATION VS DISTANCE - ALTERNATE 3 133

42 OXYGEN DEMAND VS DISTANCE - ALTERNATE 3 134

43 OXYGEN DEMAND VS DISTANCE - ALTERNATE 3 135

44 OXYGEN DEMAND VS DISTANCE - ALTERNATE 3 136

45 SCHEMATIC TREATMENT-TRANSPORT TERMINAL FACILITIES 146

46 DESIGN CONDITIONS - WASHBURN - SCHEMATIC 153

47 PRELIMINARY SITE PLAN - WASHBURN - LOWER SITE 160

48 EXPANSION - PRESQUE ISLE PLANT 163

49 PRELIMINARY SITE PLAN - PRESQUE ISLE 165

50 PRIMARY TREATMENT OPTIONS - IN-TOWN CARIBOU 171

51 PRELIMINARY SITE PLAN - CARIBOU - IN-TOWN 173

52 TRANSPORT ROUTE - GRIMES MILL TO CARIBOU 176

53 PRELIMINARY SITE PLAN - GRIMES MILL 178

54 DESIGN CONDITIONS - FORT FAIRFIELD - SCHEMATIC 180

		PAGE
55	PRELIMINARY SITE PLAN - FORT FAIRFIELD	186
56	EASTON, MAINE - PROPOSED MUNICIPAL SEWERAGE SYSTEM AND TREATMENT FACILITIES	188
57	IRRIGATION APPLICATION VERSUS ACREAGE REQUIREMENTS	194
58A & B	WASHBURN - PRESQUE ISLE CONNECTING INTERCEPTOR	197 & 198
59	MAPLETON - PRESQUE ISLE - INTERCONNECTING SYSTEM	200
60	FORT FAIRFIELD - GRIMES MILLS INTERCONNECTING ROUTE	202
61	PRELIMINARY SITE PLAN - PRESQUE ISLE - EASTON JOINT SYSTEM	205
62	PRESQUE ISLE-CARIBOU INTERCONNECTING ROUTE - SOUTHERN SECTION	208
63	PRESQUE ISLE-CARIBOU INTERCONNECTING ROUTE - NORTHERN SECTION	209
64	PRELIMINARY SITE PLAN - GRIMES MILL TERMINAL PLANT CORE AREA	212
65	TREATMENT SYSTEM DIAGRAM	298
66	AERATION UNIT CONFIGURATION - QUANTITY ESTIMATE	305
67	CORE AREA BORING LOCATIONS - SOUTHERN AREA	324
68	CORE AREA BORING LOCATIONS - NORTHERN AREA	325

TABLES

No.		Page
1	Waste Characteristic Summary - Taterstate - Raw Wastes After Screening	34
2	Waste Load Summary - Taterstate - Raw Wastes After Screening	35
3	Waste Characteristic Summary - Taterstate - Wastes After Sedimentation - Flotation	35
4	Waste Load Summary - Taterstate - Wastes After Sedimentation-Flotation	36
5	Comparison of Taterstate Wastes with Literature	36
6	Waste Characteristic Summary - Potato Service Inc - Flume Line	37
7	Waste Characteristic Summary - Potato Service Inc - Process Line	38
8	Waste Characteristic Summary - Potato Service Inc - Pond Effluent	38
9	Load Summary - Potato Service Inc	40
10	Waste Characteristic Summary - Colby Co-Op Starch Plant Raw Wastewater	41
11	Waste Characteristic Summary - American Kitchen Foods Plant - Wastes After Screening	43
12	Waste Characterization Summary - Caribou Treatment Plant - Influent & Effluent	43
13	Total Load and Unit Load Data - American Kitchen Foods	44
14	Total Load - Influent to Caribou Plant	44
15	Pea Processing Wastes - American Kitchen Foods	45
16	Waste Characteristic Summary - A & P Co - Raw Wastes After Screening	46
17	Waste Characteristic Summary - A & P Co - Wastes After Air Flotation	46

No. Page

18 Waste Load Summary - A & P Co - Raw Wastes After Screening 47

19 Waste Load Summary - A & P Co - Wastes After Air Flotation 47

20 Comparison of A & P Waste with Literature 48

21 Pea Processing - A & P Plant 48

22 Taterstate Processing Plant - Efficiency - Primary Treatment Unit 49

23 Potato Service Inc - Efficiency - Flume Water Clarifier 49

24 Potato Service Inc - Efficiency - Process Water Clarifier 49

25 Caribou Treatment Plant - Efficiency - Primary Treatment Units 50

26 A & P Processing Plant - Efficiency - Air Flotation Unit 50

27 Primary Treatment - Summary - All Units - Ave Efficiency 50

28 Town of Washburn - Domestic Waste Load 57

29 Estimated Residual Waste Load from Taterstate Based on Existing Production 60

30 Taterstate Loadings - With Flake Line Added 61

31 Town of Mapleton - Domestic Waste Load 61

32 City of Presque Isle - Domestic Waste Load 62

33 Waste Load Projections - Potato Service Inc 63

34 City of Caribou - Domestic Waste Load 64

35 Industrial Load Alternatives In-town Caribou 65

36 Fort Fairfield - Domestic Waste Load 66

37 Estimated Residual Waste Load from A & P 67

38 Easton - Domestic Waste Load 67

No.		Page
39	Industrial Loadings - Easton Area	68
40	Summary - Tentative Design Criteria - Potato Processing Wastewater Treatment	70
41	Design Loadings - Treatment-Transport System	79
42	Velocities and Water Depth for Various Channel Configurations	84
43	Channel Biological Characteristics - Minimum Load - Alternate 2	99
44	Channel Biological Characteristics - Medium Load - Alternate 2	100
45	Channel Biological Characteristics - Maximum Load - Alternate 2	101
46	Total Air Requirement - Minimum Load - Alternate 2	112
47	Total Air Requirement - Medium Load - Alternate 2	113
48	Total Air Requirement - Maximum Load - Alternate 2	114
49	Blower and Air Line Sizing - Minimum Load - Alternate 2	115
50	Blower and Air Line Sizing - Medium Load - Alternate 2	116
51	Blower and Air Line Sizing - Maximum Load - Alternate 2	117
52	Channel Hydraulic Characteristics - Minimum Load - Alternate 3	122
53	Channel Hydraulic Characteristics - Medium Load - Alternate 3	123
54	Channel Hydraulic Characteristics - Maximum Load - Alternate 3	124
55	Channel Biological Characteristics - Minimum Load - Alternate 3	128
56	Channel Biological Characteristics - Medium Load - Alternate 3	129
57	Channel Biological Characteristics - Maximum Load - Alternate 3	130

No. Page

58 Total Air Requirements - Minimum Load - Alternate 3 138

59 Total Air Requirements - Medium Load - Alternate 3 139

60 Total Air Requirements - Maximum Load - Alternate 3 140

61 Blower and Air Line Sizing - Minimum Load - Alternate 3 141

62 Blower and Air Line Sizing - Medium Load - Alternate 3 142

63 Blower and Air Line Sizing - Maximum Load - Alternate 3 143

64 Summary - Conduit Pipe Sizes - Treatment-Transport Channel - Alternate 3 144

65 BOD to MLSS Ratio - Caribou Plant Aeration 147

66 Design Condition - Coding Summary - Washburn System 154

67 Key Treatment Plant Sizing - Washburn - Condition C-1 Taterstate Plant Alone - Total Unit Size 156

68 Key Treatment Plant Sizing - Washburn - Condition C-2 Taterstate and Domestic Wastes Combined - Total Unit Size 157

69 Key Treatment Plant Sizing - Washburn - Condition C-3 Municipal Wastes Alone - Total Sizing 158

70 Design Condition - Coding Summary - Presque Isle System 161

71 Key Treatment Plant Sizing - Condition B-1 - Potato Service Inc - Potato Processing Wastes Alone - Total Unit Size 166

72 Key Treatment Plant Sizing - Condition B-2 - Combined Potato Service, Inc and Presque Isle Domestic Flows 168

73 Design Conditions - Coding Summary - Caribou In-Town System 170

74 Key Treatment Plant Sizing - Caribou 175

75 Design Condition - Coding Summary - Fort Fairfield System 179

No. Page

76 Key Treatment Plant Sizing - Fort Fairfield - Condition C-1 - A & P Plant Alone - Total Unit Size 182

77 Key Treatment Plant Sizing - Fort Fairfield - Condition C-2 - A & P and Domestic Wastes Combined Total Unit Size 183

78 Key Treatment Plant Sizing - Fort Fairfield - Condition C-3 - Municipal Wastes Alone - Total Unit Sizing 184

79 Design Conditions - Coding System - Easton Area 187

80 Key Treatment Plant Sizing - Vahlsing, Inc 190

81 Key Treatment Plant Sizing - Maine Sugar Industries 191

82 Key Treatment Plant Sizing - Easton Combined 192

83 Loading Condition Coding Summary - Presque Isle-Easton Regional System - Industry Loading 203

84 Key Treatment Plant Sizing - Presque Isle-Easton Regional Plant 206

85 Regional Treatment Plant - Caribou Area Loading Conditions 210

86 Key Treatment Plant Sizing - Caribou Regional Plant 213

87 Summary - Capital Cost Estimates - Washburn 219

88 Comparative Cost Summary - Joint vs Separate Plants - Washburn 219

89 Effect of Grant-in-Aid Programs on Local Cost - Washburn 220

90 Summary of Operating Costs - Washburn 221

91 Summary of Finance Costs - Washburn 222

92 Summary - Total Annual Revenue Requirements - Washburn 222

93 Capital Cost Estimates - Supplemental Town Facilities Washburn 223

No.		Page
94	Summary Capital Costs - Presque Isle Area	224
95	Summary - Joint System Capital Cost Savings Presque Isle Area	225
96	Local Capital Costs - Presque Isle Area	226
97	Summary - Local Capital Cost Savings - Joint System Presque Isle Area	226
98	Summary - Operating Costs - Presque Isle Area	227
99	Summary - System Finance Costs - Presque Isle Area	228
100	Summary - Annual Revenue Requirements - Presque Isle Area	228
101	Summary - Annual Cost Savings - Joint System Presque Isle Area	229
102	Capital Costs - In-Town Plant Expansion - Caribou Area	231
103	Capital Costs - Grimes Mill Site - Caribou Area	232
104	Capital Cost Differential - In-Town vs Grimes Site Caribou Area	233
105	Local Capital Costs - In-Town Plant - Caribou Area	234
106	Local Capital Costs - Grimes Site - Caribou Area	235
107	Capital Cost Comparison - Two Plant Option - Caribou Area	236
108	Summary - Operating Costs - Caribou Area	237
109	Summary - Finance Costs & Annual Revenue Requirements Caribou Area	238
110	Summary - Capital Cost Estimates - Fort Fairfield	239
111	Comparative Cost Summary - Joint vs Separate Plants Fort Fairfield	240
112	Effect of Grant-in-Aid Programs on Local Cost - Fort Fairfield	241
113	Summary of Operating Costs - Fort Fairfield	242

No.		Page
114	Summary of Finance Costs - Fort Fairfield	243
115	Summary - Total Annual Revenue Requirements - Fort Fairfield	243
116	Capital Costs - Individual Plants - Easton Area	244
117	Capital Costs - Joint Systems - Easton Area	245
118	Summary - Capital Cost Savings - Joint System Easton Area	246
119	Approximate Capital Costs - Advanced Waste Treatment Easton Area	247
120	Local Capital Costs - Easton Area	247
121	Local Capital Costs - Easton Area - Joint Municipal - Industry Systems	248
122	Summary Operating Costs - Easton Area	249
123	Summary - System Finance Costs - Easton Area	250
124	Summary - Annual Revenue Requirements - Easton Area	251
125	Interconnected Versus Community System - Washburn Area	253
126	Comparative Operating Costs - Interconnect vs Local Washburn Area	254
127	Cost Comparison - Interconnection Versus Local Treatment - Mapleton Area	257
128	Interconnected Versus Community System - Fort Fairfield to Grimes Mill	260
129	Summary Capital Costs - Joint Presque Isle-Easton	263
130	Summary - Local Capital Costs - Joint Presque Isle - Easton System	263
131	Summary Operating Costs - Joint Presque Isle-Easton System	264
132	Summary - Annual Revenue Requirements - Joint Presque Isle - Easton System	264

No. Page

133 Summary - Capital Costs - Piped Interconnect System - Core Area 265

134 Summary - Local Capital Costs - Piped Interconnect System - Core Area 266

135 Summary Operating Costs - Piped Interconnect System - Core Area 266

136 Finance Cost and Annual Revenue Requirements - Piped Interconnect System - Core Area 266

137 Summary - Capital Costs - Treatment-Transport System - Core Area 267

138 Summary - Local Capital Costs - Treatment-Transport System - Core Area 267

139 Summary - Operating Costs - Treatment-Transport System Core Area 268

140 Finance Costs & Annual Revenue Requirements Treatment-Transport System - Core Area 268

141 Piped Interconnect Versus Treatment-Transport Comparative Costs 270

142 Cost Comparisons - Local Versus Interconnected System Washburn Area 271

143 Cost Comparisons - Joint Versus Individual Plants Washburn Area 272

144 Cost Comparisons - Local Versus Interconnected System - Mapleton Area 272

145 Cost Comparisons - Local Versus Interconnected System Fort Fairfield Area 273

146 Cost Comparisons - Joint Versus Individual Plants Fort Fairfield Area 273

147 Cost Comparisons - Joint Versus Separate Treatment Presque Isle - Easton Areas 274

148 Comparative Costs - Regional Systems - Core Area 275

149 Dissolved Oxygen Conditions - Aroostook River 276

SECTION I

CONCLUSIONS

The key conclusions drawn from the Research and Development Phase of the Northern Maine Regional Treatment Project are summarized in the following items. These technical conclusions must be related to the administrative, legal, institutional, and economic conclusions contained in the companion River Basin Planning Report. The reader is referred to that report for a full discussion of the planning and water quality study phases of the Project.

GENERAL POLLUTION CONTROL CONCLUSIONS

1. Liquid wastes presently generated in the Aroostook-Prestile Basins far exceed the capacity of the rivers to receive and assimilate these wastes without extensive treatment in addition to that now provided.

2. The Aroostook River has adequate capacity for assimilating organic wastes provided treatment facilities are constructed to achieve 90% BOD removal. The most critical locations on the River will be at Caribou Dam and Tinker Dam. Present legal water quality limits, however, will be possible at these locations with recommended treatment of liquid wastes.

3. The Prestile Stream cannot assimilate the liquid wastes produced in the Easton area without utilizing advanced treatment techniques to achieve a very high BOD removal. Such techniques are very costly and are not usually considered reasonable for situations analogous to the one at hand.

4. Significant reductions in industrial liquid waste loads can be achieved by in-plant conservation and process alterations. Such reductions will not meet water quality standards, however, and additional treatment of all residual liquid wastes will be required.

5. Potato processing wastes should be subject to primary treatment for solids removal prior to biological treatment. The solids thus recovered can be utilized as animal feed.

6. Potato processing wastes can be biologically treated by the activated sludge process either separately or in conjunction with municipal wastes.

7. In all communities with potato processing industries, it is more economic to treat potato processing wastes jointly with municipal wastes than separately.

8. Disposal of excess biological solids can be a significant problem. Sanitary landfill in conjunction with municipal solid waste programs is best suited to initial operations. Effort should be made to determine the economic feasibility of utilizing the organic and nutrient value of this material as a soil conditioner.

TREATMENT-TRANSPORT SYSTEM CONCLUSIONS

9. The liquid waste treatment-transport system, wherein organic waste would be oxidized enroute in an eleven mile regional facility between Presque Isle and Caribou, was shown to be technically feasible.

10. The treatment-transport system would require active aeration sources because the hydraulic grade and surface area of the system could not meet the oxygen demand requirements of the biological system through natural means.

11. In order to obtain BOD removals in excess of 85% in the treatment-transport system, extension solids return facilities are required. The solid return facilities could be eliminated if sufficient quantities of an active biological mass were introduced at the head end of the system.

12. The liquid waste treatment-transport system, was shown to be roughly equal in cost to conventional pipeline transfer followed by more extensive treatment at the terminus. Therefore, there is no clear advantage to the treatment-transport system over more conventional piped transport methods.

13. To assess the applicability of the treatment-transport system to other pollution control systems, additional studies are necessary concerning the economic effects of shorter or longer distances, greater or lesser changes in elevation, and volume and strength of wastes to be treated.

POLLUTION CONTROL FACILITY CONCLUSIONS

14. Detailed design and cost analyses have shown that a completely interconnected liquid waste system is uneconomical for the Northern maine region and would not achieve the objective of minimizing costs. This is primarily due to the wide separation and relatively small number of major waste sources and high transportation costs.

15. Treatment of municipal and industrial wastes generated in the Towns of Washburn, Mapleton, and Fort Fairfield at facilities located within those communities is more economic than transporting such wastes for treatment at a downstream location.

16. The least cost alternative for treatment of liquid wastes generated in the Easton area is to transfer them to the Aroostook River Basin for treatment in conjunction with Presque Isle's liquid wastes. A reverse transfer of fresh water will be necessary to maintain adequate flow in the Prestile Stream during times of extreme low flow. Such interbasin transfer will require enabling legislation.

17. The Prestile Stream can be returned over time to a condition which will support a sport fishery if liquid wastes generated in the Easton area are removed from the Basin.

18. Within the Core Area of Presque Isle, Caribou, and Easton, three alternative means of waste treatment are feasible:

    a. A two plant system with treatment facilities located in both Presque Isle and Caribou,
    b. a single treatment plant in Caribou with Presque Isle and Easton wastes transported to the Caribou facility in a conventional interceptor pipeline, and
    c. a single treatment plant in Caribou with Presque Isle and Easton wastes transported to the Caribou facility in an eleven mile treatment-transport system wherein organic wastes would be oxidized enroute.

19. If the two plant system is adopted in the Core Area, the Caribou facilities could consist of either an expanded in-town plant providing both primary and secondary treatment, or the existing in-town plant for primary treatment supllemented by a biological treatment plant located at Grimes Mill.

20. The in-town plant site is adequate to provide secondary treatment to local waste loads, but inadequate area exists to enable expansion for advanced waste treatment facilities.

21. Secondary treatment at the Grimes Mill site would be somewhat more costly than expansion of the in-town plant, but offers the advantage of providing the area required for major treatment plant expansion to accomodate additonal industrial flows and/or advanced waste treatment, if required in the future.

22. If the one plant system utilizing the conventional pipe interceptor transport is adopted for the Core Area, the treatment facility should be constructed at the Grimes Mill site due to side limitations at the existing in-town facility.

23. The capital cost for the one plant facility is approximately 19% higher than the two plant system. However, operating costs are slightly less.

24. The one plant system offers several ancillary benefits over the two plant system resulting in improved water quality between Presque Isle and Caribou and the benefits derived therefrom, including outdoor recreation and fisheries, improvement of Caribou's municipal water supply, a larger and more economical service area, a stimulant to economic growth, and efficiency in the potential reuse of solids.

25. The international nature of the Aroostook River and Prestile Stream render ultimate water quality requirements subject to international agreement with potentially significant effects upon long range treatment methods. Should such agreements require higher dissolved oxygen concentration, or if higher loads are generated, additional measures may be required during low flow periods in the future, such as effluent holding ponds, advanced waste treatment, in-stream aeration, use of pure oxygen, production curtailment, or a combination thereof. The installation of drought condition effluent holding ponds, or use of pure oxygen for aeration of the effluent appear the most economic solutions.

SECTION II

RECOMMENDATIONS

The following technical recommendations resulted from the research and development phase of the Northern Maine Regional Treatment Project. It should be noted that several of the technical recommendations must be reviewed by the Commission upon evaluation of the administrative, legal, institutional, and economic recommendations as presented in companion River Basin Planning Report. The reader is referred to that report for detailed discussions and recommendations concerning the planning and water quality study phases of the Project.

GENERAL RECOMMENDATIONS

1. A system of pollution abatement facilities is needed to meet water quality standards within the Aroostook-Prestile Basin.

2. Local industry must evaluate their internal operations in light of the data presented in the companion Research and Development Report, and should determine the residual waste load to be treated externally after in-plant water conservation.

3. All potato processing wastes must receive primary treatment or its equivalent solids removal prior to entry into the biological system.

4. Removed potato solids should be utilized as an animal feed, either directly or through further processing.

5. Biological treatment systems should be designed to remove 90% of the residual BOD at 20°C.

6. Attempts to develop a soil conditioning program utilizing the excess biological solids from the treatment facilities should be implemented with the necessary additional research being undertaken as soon as possible. Until this is accomplished the excess biological solids should be disposed of in conjunction with municipal solid waste disposal program.

POLLUTION CONTROL FACILITY RECOMMENDATIONS

7. Washburn - A treatment plant should be constructed at Washburn serving both the Washburn domestic and the Taterstate processing wastes. Discharge should be to the Aroostook River via a diffuser system rather than to Salmon Brook.

8. Mapleton - An extended aeration treatment plant should be constructed at Mapleton to treat the domestic waste flows.

9. Fort Fairfield - A treatment plant should be constructed at Fort Fairfield serving both the Fort Fairfield domestic wastes and the A & P processing wastes.

10. Easton - Wastewaters generated in Easton should be transferred to the Aroostook River Basin for treatment in conjunction with other Core Area wastes from Presque Isle and Caribou.

11. Core Area - The Core Area of Presque Isle, Caribou, and Easton should be served by an interconnected one plant system in the plant located at the Grimes Mill site. This recommendation is based upon long range cost minimization, the ancillary benefits to be derived, and feasibility for future expansion of the facility and/or the provision of advanced waste treatment.

    Pollution control facilities installed in the Core Area must include provision for future installation of effluent diversion and holding ponds, and space for future installation of advanced waste treatment facilities.

    The interconnected one plant system for the Core Area should use the conventional closed pipe system. The treatment-transport system is not recommended due to greater cost and the need for large scale pilot plant studies to confirm design data which could measurably delay the project.

IMPLEMENTATION RECOMMENDATIONS

12. Implementation of the recommendations of the report must be undertaken as expeditiously as possible to meet the State requirement that all facilities be in operation by October, 1976.

13. A special session of the Maine legislature should be requested so that enabling legislation for the river basin authority and interbasin transfers between the Aroostook River and Prestile Stream can be enacted in 1972.

14. Final plans for recommended facilities must be commenced during 1972 and completed no later than April, 1974.

15. Construction of facilities must be commenced no later than May, 1974.

ACTION TAKEN BY NORTHERN MAINE REGIONAL PLANNING COMMISSION

The following resolutions were adopted at a meeting of the Executive Committee of the Board of Directors of the Northern Maine Regional Planning Commission, held in the Commission's offices on March 16, 1972:

1. To accept the Northern Maine Regional Treatment System Research and Development Report and the River Basin Planning Report as submitted by Edward C. Jordan Co, Inc, subject to revisions as requested by the Environmental Protection Agency and the Maine Environmental Improvement Commission.

2. To adopt as the Regional Plan, the recommendations proposed by the Jordan Company, specifically including:

    a. Separate treatment facilities to serve Mapleton, Washburn and Taterstate, and Fort Fairfield and A & P.

    b. A regional treatment facility to serve the Core Area of Presque Isle, Caribou, and Easton consisting of an interconnected piped system and a single treatment facility to be located at Grimes Mill.

3. To seek creation of an interbasin river basin authority with the following powers:

    a. Right of eminent domain.

    b. To effect interbasin transfers of water and liquid wastes.

    c. To finance and construct regional facilities.

    d. To set rates on a relative use basis.

    e. To collect revenues from users.

    f. To require curtailment of industrial discharges during critical low flow periods.

    g. To purchase or take existing treatment facilities.

    h. To approve and coordinate all treatment facilities within the Basins.

    i. To enter into contracts with municipalities for the maintenance of collector sewers.

    j. To sponsor soil conservation projects.

    k. To accept grant-in-aid funds.

    l. To require mandatory participation in the regional pollution control system.

    m. To exercise other authority necessary for implementation of the Regional Plan.

SECTION III

INTRODUCTION

Aroostook County is located in the Northern extremity of the State of Maine, bounded on the east by the Province of New Brunswick and on the west by the Province of Quebec, Canada. Nearly all of the natural drainage from the County is tributary to the international Saint John River. One of the major sub-basins of the Saint John Basin is the Aroostook River which drains the central area of the County. The Prestile Stream sub-basin is a minor segment of the Saint John River system, lying adjacent to the larger Aroostook River Basin. The general geographical orientation of the region is shown on Figure 1.

The Aroostook River Basin contains the major population and industrial centers of northern Maine, with main population centers at Fort Fairfield, Caribou, and Presque Isle. The economy of the Basin is strongly agriculturally oriented, with potatoes being the dominant crop. Agriculture is concentrated in the lower reaches of the Basin, while much of the headwater areas are wild lands in timber management.

Since the early 1950's the potato economy of the nation, and Aroostook County, has evolved from predominantly a fresh pack market to a substantial processed product market. Currently about 40% of the Aroostook crop is processed, primarily into frozen french fries and by-products. The trend toward processing a larger portion of the crop is expected to continue into the 1970's.

Processing of a substantial portion of the potato crop has created an increasingly serious pollution problem in the Aroostook River and adjacent Prestile Stream. The organic waste loads now generated by potato processing plants in the area represent a load nearly thirty times that generated by the resident population. This condition has created nuisance conditions in both the Aroostook and Prestile waters, and represents a frequent violation of State and Federal water quality standards.

In the mid 1960's it became evident that further expansion of the processing industries of northern Maine was vitally dependent upon finding a solution to the severe pollution problems of the area, consistent with the economic resources of the communities and industries of the Basin. To accomplish this objective the Northern Maine Regional Planning Commission initiated a preliminary evaluation of the problem which culminated in a general comprehensive water resources plan for the Aroostook-Prestile Basins[34]. This initial report recommended that the Aroostook-Prestile Basins be considered as a hydrologic unit, and that all pollution control efforts be coordinated on a river basin planning basis. The report also suggested that physical integration of facilities, particularly those serving the upper Prestile Basin, could achieve the maximum degree of water quality improvement at minimum cost. These early studies indicated that the geographical and topographic configuration of the lower Aroostook Basin may permit utilization of a combined treatment-transport system to achieve regional system economies.

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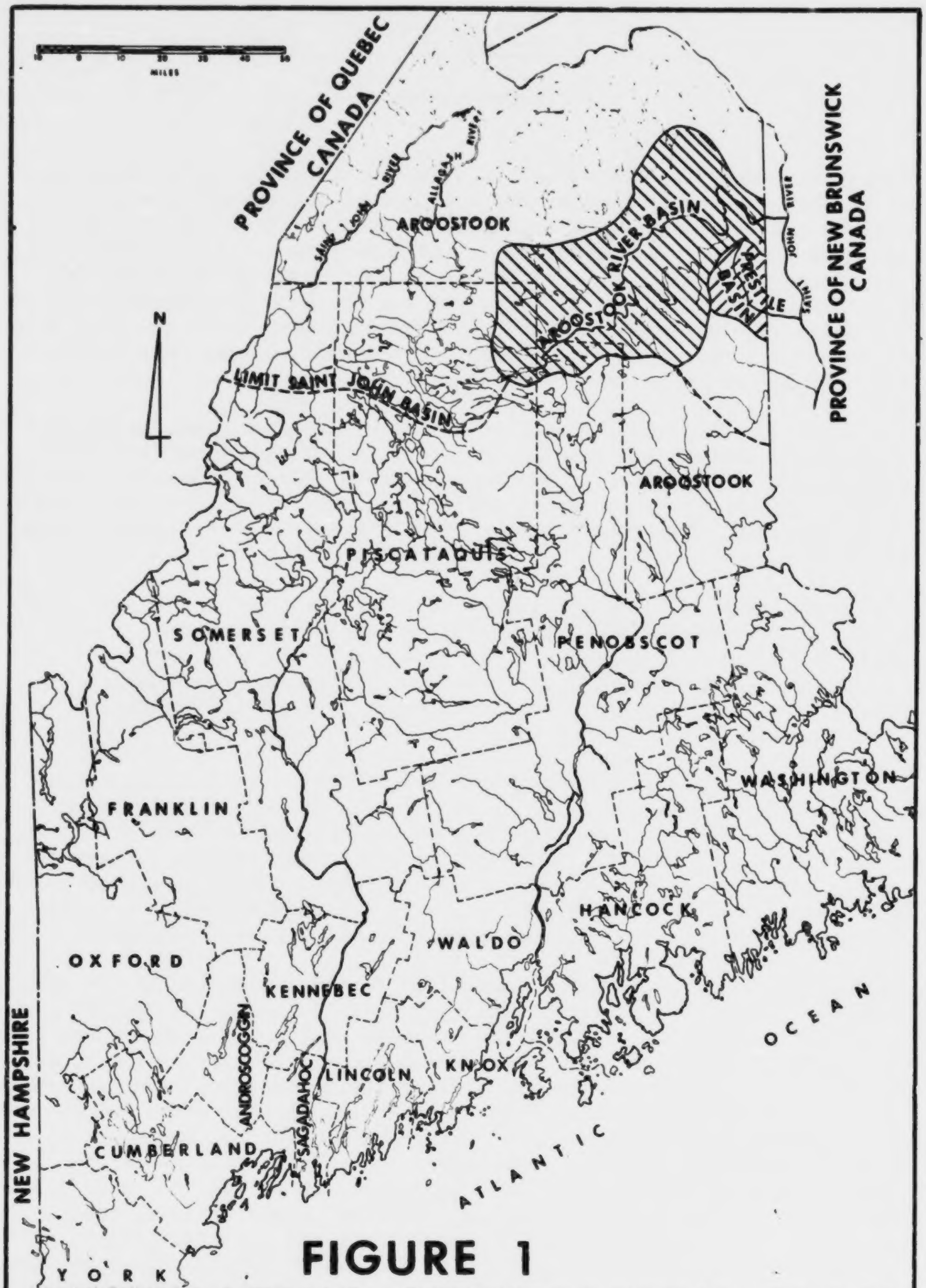
MILES
PROVINCE OF QUEBEC
CANADA
N
SAINT JOHN RIVER
ALLAGASH RIVER
AROOSTOOK
AROOSTOOK RIVER BASIN
PRESTILE BASIN
SAINT JOHN RIVER
PROVINCE OF NEW BRUNSWICK
CANADA
LIMIT SAINT JOHN BASIN
AROOSTOOK
PISCATAQUIS
SOMERSET
PENOBSCOT
WASHINGTON
FRANKLIN
HANCOCK
WALDO
OXFORD
KENNEBEC
ANDROSCOGGIN
NEW HAMPSHIRE
SAGADAHOC
LINCOLN
KNOX
CUMBERLAND
ATLANTIC
OCEAN
YORK

FIGURE 1
GEOGRAPHIC ORIENTATION PLAN

The Northern Maine Regional Planning Commission adopted the recommendations of the initial report as the basis for subsequent pollution control planning in the Aroostook-Prestile Basins. The Commission retained the Edward C. Jordan Company, Consulting Engineers and Planners, to organize an extensive planning effort which would lead to a Basin-wide program of pollution abatement. The proposed preliminary design and planning effort was divided into two basic phases, research and development, and Basin planning; with each closely coordinated with the other to assure implementation of the program at the earliest possible date.

A critical-path-method flow diagram and schedule was prepared for the engineering and planning work to be accomplished. This flow diagram is shown on Figure 2. As indicated on the figure, the research and development phase of the project was directed at the many alternatives available for regional treatment of the industrial, or combined industrial-domestic waste loads. The primary consideration in this analysis was the research and development phase for a process establishing the feasibility of constructing a treatment-transport system for treating the wastes while in transit within an interconnected regional treatment facility. It is noted that the original program included a pilot plant segment under the research and development phase. This work was subsequently omitted from the research and development program. In order to determine the economic feasibility of the treatment-transport system, for the area under consideration, it was necessary to establish potential design loadings and preliminary design for alternative conventional treatment systems.

The river basin planning phase of the project consisted of three segments; namely, river and water quality studies, economic and land use studies, and studies of administrative and financial systems necessary for project implementation on a basin-wide basis. By closely coordinating both phases of the study, it will be possible to shorten the time lag normally occurring between completion of research and development studies and actual pollution abatement. The CPM diagram of Figure 2 shows the interplay of both phases of the project.

Application was made to the Environmental Protection Agency for funding of the program under Section 5, Research and Development, and Section 3C, River Basin Planning, of the Federal Water Pollution Control Act, as amended. Local funding was arranged through a combination of substantial input from local industries and the State of Maine. The work was funded in late 1970 as Project 16110 DPT, Research and Development, and Project IGA 00021, River Basin Planning. Such a combination of effort is unique in water resource planning.

The overall report will be published in two parts representing the two major phases of work. This report presents the work accomplished under the Research and Development Phase, while the second report will present the Basin Planning efforts. Due to the close relationship and interplay of the two phases, it is recommended that the reader examine both reports to assure full appreciation of the implications of each for the other.

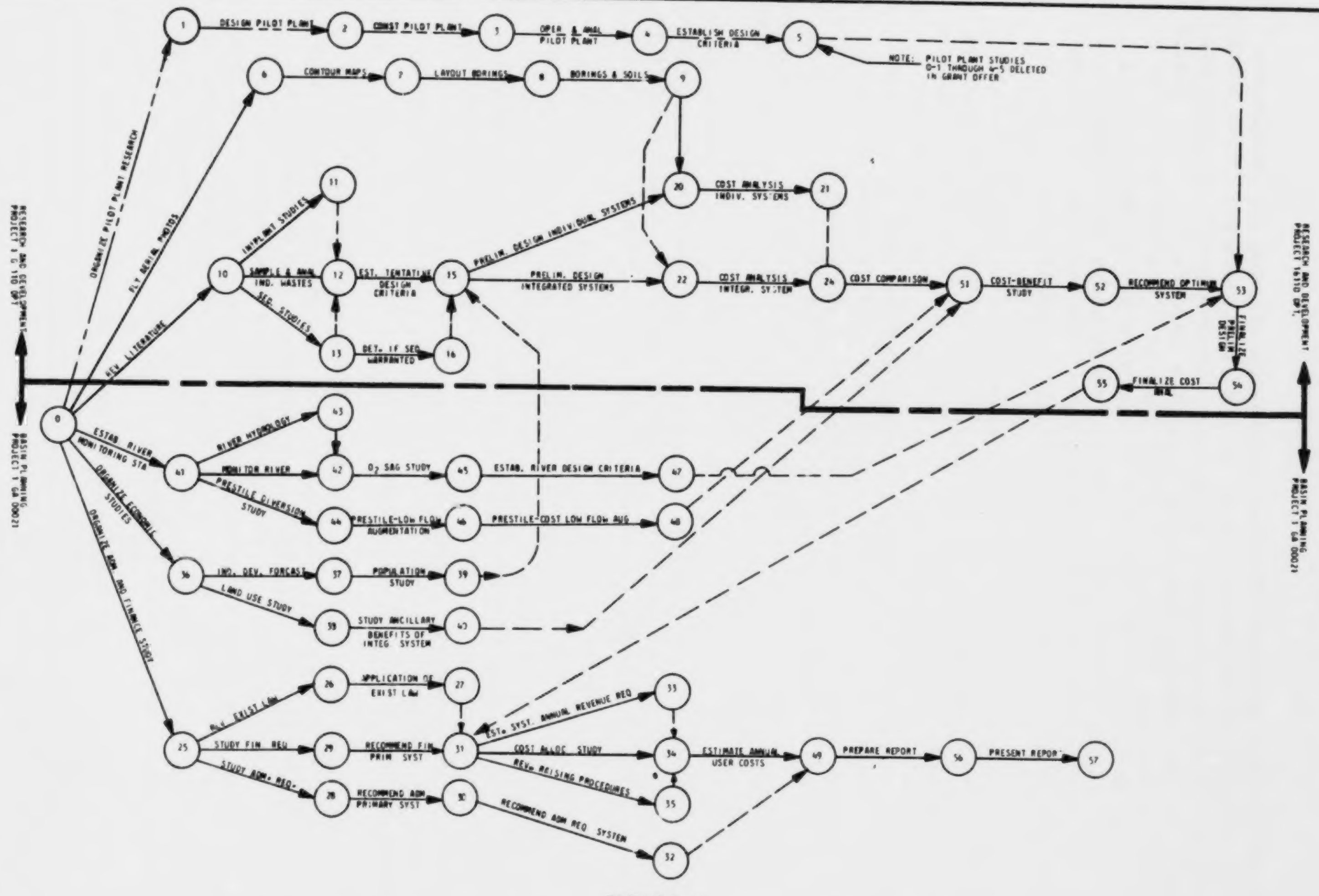
RESEARCH AND DEVELOPMENT PROJECT 1 G 110 DPT
BASIN PLANNING PROJECT 1 GA 00021
RESEARCH AND DEVELOPMENT PROJECT 16110 DPT.
BASIN PLANNING PROJECT 1 GA 00021
ORGANIZE PILOT PLANT RESEARCH
FLY AERIAL PHOTOS
REV. LITERATURE
ESTAB. RIVER MONITORING STA
ORGANIZE ECONOMIC STUDIES
ORGANIZE ADM AND FINANCE STUDY
DESIGN PILOT PLANT
CONST PILOT PLANT
OPER. & ANAL PILOT PLANT
ESTABLISH DESIGN CRITERIA
NOTE: PILOT PLANT STUDIES 0-1 THROUGH 4-5 DELETED IN GRANT OFFER
CONTOUR MAPS
LAYOUT BORINGS
BORINGS & SOILS
INPLANT STUDIES
SAMPLE & ANAL IND. WASTES
SEQ. STUDIES
EST. TENTATIVE DESIGN CRITERIA
DET. IF SEQ WARRANTED
PRELIM. DESIGN INDIVIDUAL SYSTEMS
PRELIM. DESIGN INTEGRATED SYSTEMS
COST ANALYSIS INDIV. SYSTEMS
COST ANALYSIS INTEGR. SYSTEM
COST COMPARISON
COST-BENEFIT STUDY
RECOMMEND OPTIMUM SYSTEM
FINALIZE PRELIM DESIGN
FINALIZE COST ANAL
RIVER HYDROLOGY
MONITOR RIVER
PRESTILE DIVERSION STUDY
O2 SAG STUDY
ESTAB. RIVER DESIGN CRITERIA
PRESTILE-LOW FLOW AUGMENTATION
PRESTILE-COST LOW FLOW AUG
IND. DEV. FORCAST
POPULATION STUDY
LAND USE STUDY
STUDY ANCILLARY BENEFITS OF INTEG SYSTEM
ALL EXIST LAW
APPLICATION OF EXIST LAW
STUDY FIN REQ
RECOMMEND FIN PRIM SYST
STUDY ADM. REQ.
RECOMMEND ADM PRIMARY SYST
EST. SYST. ANNUAL REVENUE REQ
COST ALLOC STUDY
REV. RAISING PROCEDURES
RECOMMEND ADM REQ SYSTEM
ESTIMATE ANNUAL USER COSTS
PREPARE REPORT
PRESENT REPORT
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FIGURE 2

THE NORTHERN MAINE REGIONAL TREATMENT PROJECT CRITICAL PATH METHOD FLOW DIAGRAM

The combined studies and data presented in the two reports, Research and Development, and Basin Planning, meet the requirements of both the river basin planning and metropolitan/regional planning studies contained in Regulation 18 CFR 601.32 & 33.

SECTION IV

INDUSTRIAL WASTE STUDIES

The major pollution loads generated in the Aroostook-Prestile Basins are industrial in nature, primarily from the food processing industry. The food processing industry is very heavily oriented to the major crop, potatoes. Relatively minor processing inputs are generated from pea packing and the refining of sugar from beets, although this latter industry has not as yet functioned to capacity to allow full evaluation of its impact on water quality.

Prior to undertaking an evaluation of the research and development studies for the abatement program, an evaluation of the existing and projected waste loads from the Basin's major industrial plants was necessary. This work was accomplished by a rather extensive literature review and a program of in-plant gaging, sampling, and analysis, where possible. A summary of the literature review is presented in Section XI with a reference listing. The results of the on-site testing and evaluation programs are presented in the following sub-sections.

IN-PLANT STUDIES

The major industrial plants in the study area are listed below with location and product designation. The approximate plant locations are shown on Figure 3.

Taterstate Frozen Foods, Washburn: Frozen french fries and by-products.
Potato Service, Inc, Presque Isle: Frozen french fries and by-products, and potato flakes.
American Kitchen Foods, Caribou: Frozen french fries and by-products.
Colby Cooperative Starch Co, Caribou: Potato starch.
Cyr Brothers Meat Packaging, Inc, Caribou: Frozen french fries and by-products.
A & P Co, Fort Fairfield: Frozen french fries and by-products, and peas.
Vahlsing, Inc, Easton: Frozen french fries and by-products.
Maine Sugar Industries, Easton: Refined sugar.

In-plant inspections and process studies preceded actual gaging and sampling programs at all of the above plants except American Kitchen Foods, Vahlsing, Inc, and Maine Sugar Industries. General process review and suggestions for in-plant waste load reductions are given in the following paragraphs.

TATERSTATE FROZEN FOODS - The Taterstate Frozen Foods plant at Washburn produces frozen french fries and by-products. This plant is typical of

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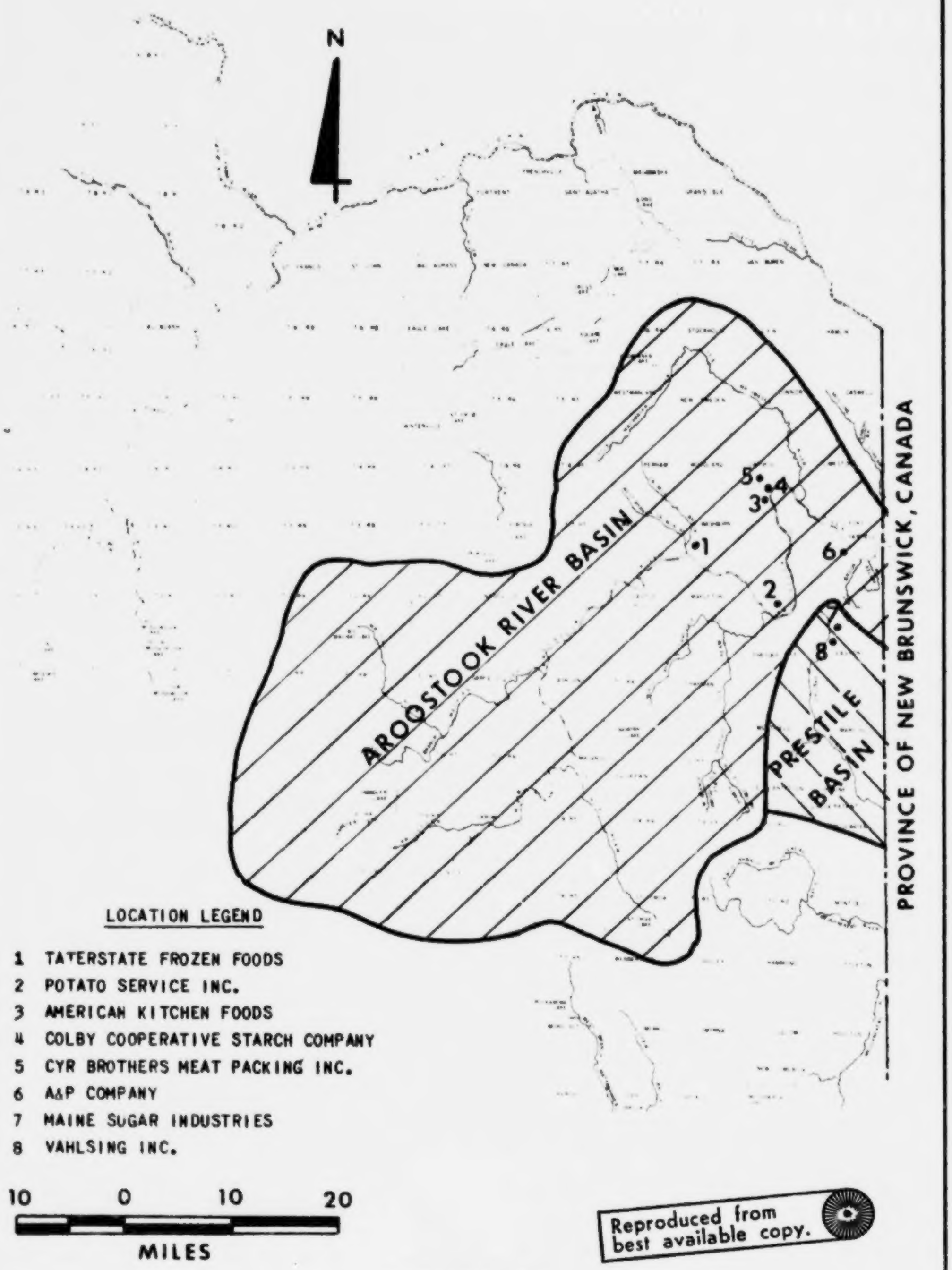
N
AROOSTOOK RIVER BASIN
PRESTILE BASIN
PROVINCE OF NEW BRUNSWICK, CANADA
1
2
3
4
5
6
8
LOCATION LEGEND
1 TATERSTATE FROZEN FOODS
2 POTATO SERVICE INC.
3 AMERICAN KITCHEN FOODS
4 COLBY COOPERATIVE STARCH COMPANY
5 CYR BROTHERS MEAT PACKING INC.
6 A&P COMPANY
7 MAINE SUGAR INDUSTRIES
8 VAHLSING INC.
10 0 10 20
MILES

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FIGURE 3

INDUSTRY LOCATION PLAN

plants utilizing the steam peel process. A generalized schematic process diagram of the Taterstate operation is shown in Figure 4. This diagram represents plant conditions at the time of gaging and sampling in the spring of 1971, and may not represent adjustments made since that time to effect a waste reduction program.

Potatoes are transported from storage to the process area via a fluming system which also serves as the main washing mechanism. The flume facility uses a recycle system to the extent possible, with silt and mud removed periodically by hand. The potatoes are peeled by the steam peel method with some additional peeling by abrasion. The peeled potatoes are then cut, sorted and blanched prior to frying. After frying the product is cooled, quick frozen, and packaged for storage. Slivers, nubbings and other edible rejected pieces are utilized in a frozen by-products line. Process water is taken from Salmon Brook, a tributary of the Aroostook River. The intake water is first utilized for refrigerationn purposes, after which the required process water is taken into the plant, while the excess is returned to the brook.

Wastewater generation in the spring of 1971 amounted to about 1.0 million gallons per day (MGD), with a raw potato input of about 340 tons per day. This represents a process water usage of about 3000 gallons per ton of potato input. This is somewhat below the reported industry average and suggests relatively good utilization of water within the plant.

The waste stream passes through a screening unit and a sedimentation unit prior to discharge to Salmon Brook. The solids thus removed are transferred to Presque Isle for processing into an animal feed product. The effectiveness of the existing treatment units is discussed in a later section.

A review of the Taterstate operation in the spring of 1971 suggested several procedures for consideration by management for reducing waste loads prior to external treatment. The suggested in-plant alterations are briefly described as follows:

1. Review all existing in-plant recycle systems to maximize water reuse and minimize net outflow.
2. Install automatic valves on general use hoses and conduct in-plant education on their use.
3. Intercept all piped systems, including boiler water, to the central waste system to eliminate miscellaneous discharges, however minor they may be.
4. Install a finer mesh screen on the existing screening facility.
5. Install dry caustic peel procedures to replace existing peeling equipment.
6. Utilize cyclone type solids removal units on certain waste streams, to remove concentrated protein water for salvage.

Items 1, 2, and 3, above, are for the most part routine housekeeping suggestions. Their implementation may not reduce flows greatly, but

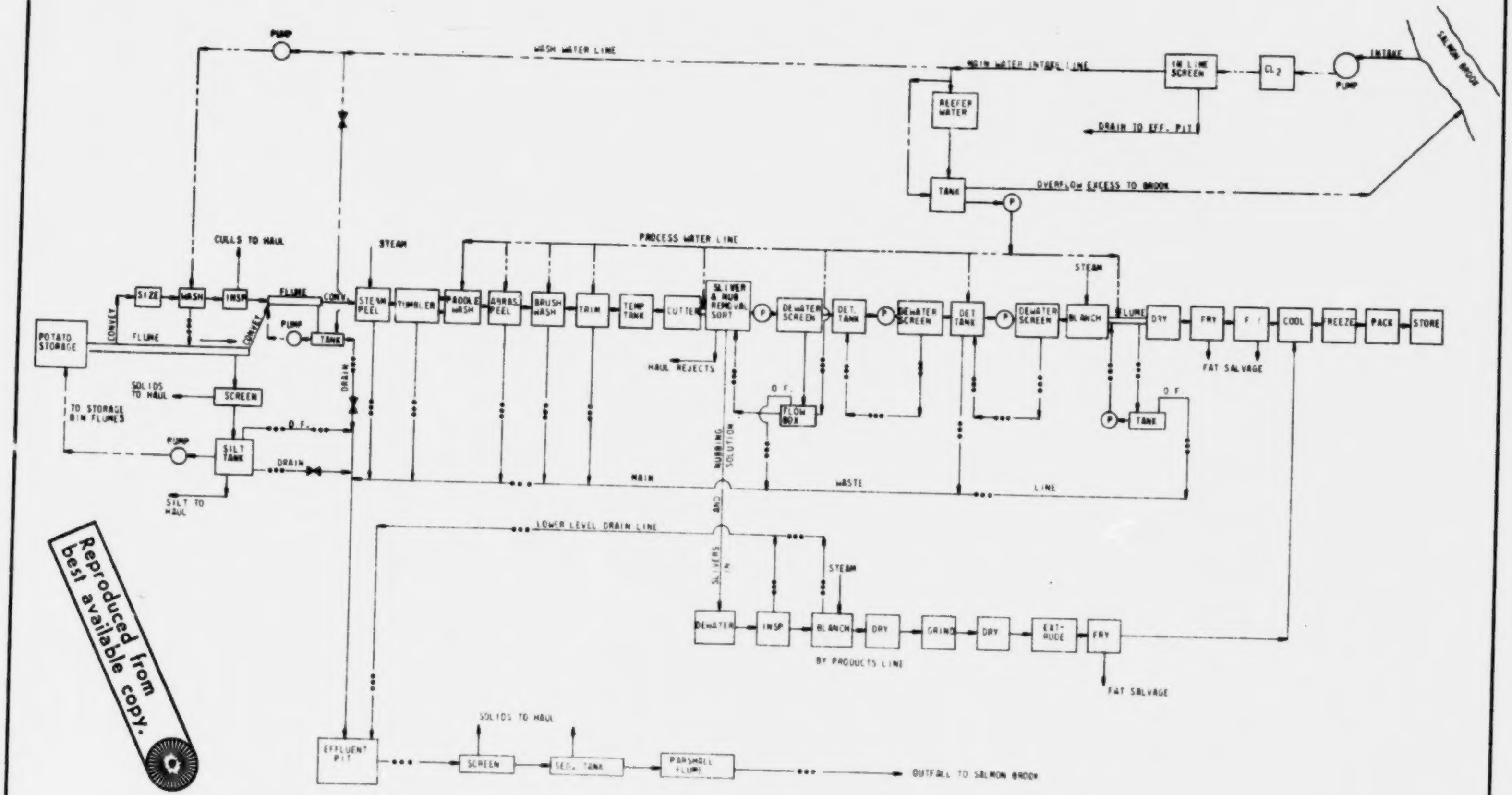
SALMON BROOK
INTAKE
PUMP
CL 2
IN LINE SCREEN
MAIN WATER INTAKE LINE
DRAIN TO EFF. PIT
REEFER WATER
TANK
P
OVERFLOW EXCESS TO BROOK
WASH WATER LINE
PUMP
PROCESS WATER LINE
STEAM
CULLS TO HAUL
POTATO STORAGE
CONVEY
FLUME
SIZE
WASH
INSP
FLUME
CONVEY
CONV.
PUMP
TANK
STEAM PEEL
TUMBLER
PADDLE WASH
ABRAS. PEEL
BRUSH WASH
TRIM
TEMP TANK
CUTTER
SLIVER & NUB REMOVAL SORT
HAUL REJECTS
P
DEWATER SCREEN
DET. TANK
P
DEWATER SCREEN
DET TANK
P
DEWATER SCREEN
STEAM
BLANCH
FLUME
DRY
FRY
F F
COOL
FREEZE
PACK
STORE
FAT SALVAGE
O F.
FLOW BOX
O F
P
TANK
SOLIDS TO HAUL
SCREEN
TO STORAGE BIN FLUMES
PUMP
SILT TANK
O F.
DRAIN
DRAIN
SILT TO HAUL
MAIN
WASTE
LINE
NUBBING SOLUTION
AND
SLIVERS IN
LOWER LEVEL DRAIN LINE
STEAM
DEWATER
INSP
BLANCH
DRY
GRIND
DRY
EXT-RUDE
FRY
BY PRODUCTS LINE
FAT SALVAGE
EFFLUENT PIT
SOLIDS TO HAUL
SCREEN
SED. TANK
PARSHALL FLUME
OUTFALL TO SALMON BROOK

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FIGURE 4 SCHEMATIC PROCESS FLOW DIAGRAM TATERSTATE-WASHBURN

will assure that flows are maintained at consistent levels. Unless these items are continually reviewed and checked, there will tend to be a gradual increase in plant water usage, as employees find it more convenient to use excessive water. Most of the plant wastes now discharge through the main drain. However, boiler water and possibly other small drains still discharge to the bank area adjacent to the plant. To assure that all wastes are adequately controlled, these discharges must be brought into the main drain for measurement and treatment.

Item 4 suggests increasing the solids and BOD removals by using a finer mesh screen in the screening unit. At the time of the testing program, a 4 mesh unit was in use. Mesh sizes of 20 to 40 are in common use. It is recognized that the finer mesh units require more maintenance and cleaning than do open screens, but it is much less costly to remove BOD by screening than by secondary treatment.

Item 5 above considers replacement of the existing peeling equipment with a dry caustic peel line, with semi-dry haul of peel wastes. Such equipment has been developed and is in current use in several plants, mostly in the western states. Operational evaluation of this type of equipment is currently being undertaken by processors and through studies supported by funds of the Environmental Protection Agency. While these extensive evaluation studies are not complete, data to date suggests a significant reduction in waste load can be accomplished by utilizing dry caustic peel equipment. This installation, however, must be considered a major plant alteration with substantial investment required.

Item 6 suggests the possibility of removing concentrated protein water from certain waste streams within the plant by use of centrifugal, or cyclone type, solids concentration units. Limited work with these units within the industry suggest that significant organic waste reductions may be achieved. The economics of the process will depend on a market for the salvaged materials and on the haul distance to that market.

The potential for residual-waste-load reduction through the above alterations is presented in a later section of this report. Prior to final design of pollution abatement facilities at Washburn, the management of the Taterstate plant must evaluate its operation to determine the waste-load reductions which can be achieved by process changes and pre-treatment, and the residual waste loads which must be handled externally and paid for by the Company.

POTATO SERVICE, INC - The Potato Service, Inc, processing plant in Presque Isle is the largest in Maine, and one of the largest in the country, with a potato intake capacity of over 1000 tons per day. Potatoes are processed into frozen french fries, frozen by-products, and potato flakes. A generalized schematic process diagram is shown in Figure 5. It is emphasized that this diagram does not reflect every function in detail, and is presented to provide a general overview of the process.

The potatoes are transferred from storage to the process unit by a

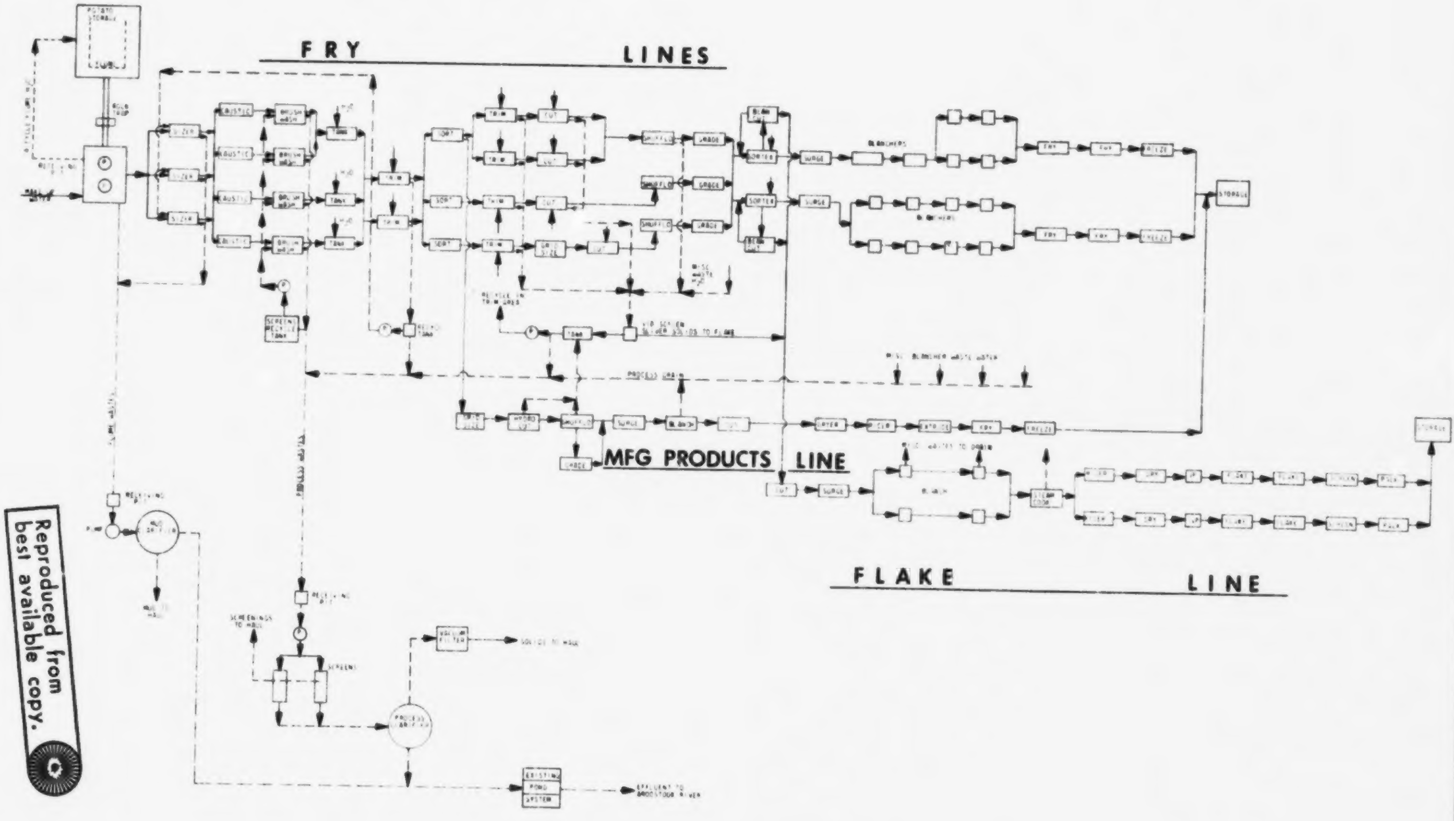
FRY LINES
MFG PRODUCTS LINE
FLAKE LINE
POTATO STORAGE
FLUME
RECEIVING PIT
SIZER
CAUSTIC
BRUSH WASH
TANK
H2O
TRIM
SORT
CUT
GRID SIZE
SHUFFLE
GRADE
SORTER
SURGE
BLANCHERS
FRY
FREEZE
STORAGE
SCREENS RECYCLE TANK
RECYCLE TANK
RECYCLE IN TRIM AREA
VIB SCREEN SILVER SOLIDS TO FLAKE
MISC WASTE H2O
PROCESS DRAIN
MISC BLANCHER WASTE WATER
HYDRO CUT
BLANCH
DRYER
EXTRUDE
STEAM COOK
MISC WASTES TO DRAIN
DRY
FLAKE
SCREEN
PACK
FLUME WATER
PROCESS WATER
PUMP
MUD CLARIFIER
MUD TO HAUL
SCREENINGS TO HAUL
SCREENS
PROCESS CLARIFIER
VACUUM FILTER
SOLIDS TO HAUL
EXISTING POND SYSTEM
EFFLUENT TO AROOSTOOK RIVER

FIGURE 5 SCHEMATIC PROCESS DIAGRAM POTATO SERVICE INC.

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fluming system utilizing partial recycling of flume water. The potatoes are then sized and peeled by the standard lye peel process. The peeled potatoes are trimmed, sorted and cut prior to blanching. The blanched product is fried, cooled and quick frozen for storage. The trimmings, slivers and other edible waste pieces are utilized in a frozen by-products line, or a potato flake line. Process water is taken from a series of wells adjacent to the Aroostook River.

Wastewater generation in the spring of 1971 amounted to about 3.1 MGD, of which about 0.5 MGD was attributable to the fluming system and about 2.6 MGD to the various process lines. This flow represents a water usage of just over 3000 gallons per ton of raw potato input. This is somewhat below the reported industry average.

The waste streams from the fluming system and from the process line are each given primary treatment in separate units. The flume wastewaters pass through a clarifier at the plant, and the removed mud is truck hauled. The process waste stream is screened and passed through a relatively new primary clarifier. The solids from this clarifier are dewatered on a vacuum filter. The solids from both the screening units and the primary clarifier are hauled for use as animal feed.

After primary clarification the waste streams are combined and pass to a pond system prior to discharge to the Aroostook River. The pond system is three celled, with the initial cell equipped with some aeration capacity. The effectiveness of the existing treatment units is discussed in a later section.

A review of the Potato Service, Inc, operation in the spring of 1971 suggested several procedures for consideration by management for reducing waste loads prior to external treatment. The suggested in-plant alterations are briefly described as follows:

1. Review all existing in-plant recycle systems to maximize water reuse, with special attention to additional recycling of the mud clarifier effluent to the flume system.
2. Install automatic valves on general use hoses and conduct in-plant education on their use. Consideration should be given to better automatic flow controls to eliminate excessive floor spillage.
3. Use cyclone type solids removal units on certain waste streams, to remove concentrated protein water for salvage.
4. Install dry caustic peel procedures to replace existing peeling equipment.

Items 1 and 2 above are for the most part housekeeping in nature. The existing mud clarifier is quite effective for silt removal from the flume wastes. It would seem possible to recycle more of this effluent within the flume, and thus reduce residual flume system wastes. Process control of the plant's complex production lines appears to be a problem, with frequent floor spillages occuring because of surges. Improved flow controls on the system could reduce these waste flows to some extent.

Item 3 suggests the installation of cyclone-type solids concentrators on certain waste streams. Such facilities have the potential for increased in-plant organics removal, with production of a possibly valuable by-product for starch recovery. Testing of this type of equipment was underway during the course of this study, and appears to show promise.

Item 4 above considers replacement of the existing peeling equipment with dry caustic peel systems, with semi-dry haul of peel wastes. The comments on application of this equipment to the Potato Service, Inc, plant are similar to those previously presented for Taterstate.

The potential for residual-waste-load reduction through implementation of the above alterations is presented in a later section of this report. Prior to final design of pollution abatement facilities in the Presque Isle area, management must evaluate its operation to determine the reductions in waste load attainable by process changes and pre-treatment, and the residual waste loads which must be handled at the Company's expense.

CYR BROS MEAT PACKING, INC - Cyr Bros Meat Packing, Inc, is a Caribou firm which has operated in the meat packing field for a number of years. As no slaughtering is accomplished on site, its wastewater loads have been limited, and have been discharged to the Caribou Utilities District system. In 1970 the firm began planning to enter the potato processing field with french fry, by-products and whole white product lines. The plant expansion has been under construction during this study.

The Cyr Bros plan has been developed around the dry caustic peel system, with intensive in-plant water recycle. A schematic process flow diagram is shown on Figure 6. The potatoes are transferred from storage to processing by a water flume system. The flume water will be reconditioned by screening and a cyclone-type cleaner, and recycled through the system. The solids will be removed by haul, and the excess flow will be disposed in a subsurface disposal system.

The potatoes will be peeled by the dry caustic system, with the dry scrubbings carried to a solids hopper. The final wash of the peeled potatoes will be recycled after screening and conditioning with centri-cleaners. The peeled potatoes will be water carried to the trimming and cutting areas, with the carriage water recycled through the system after conditioning with sieve and centri-cleaner units. Solids removed from this system will be carried to the solids hopper. Slivers, etc, will be taken from the trim and cut operations for by-product processing. The potatoes will then be blanched, fried, cooled and frozen in normal procedures.

A floor flume system will be provided to collect all drippings, spillages and wash downs. This system will enter a receiving hopper where the solids concentrated at the bottom will be pumped to a hydrosieve installation. The solids will enter the collection hopper. The liquid component

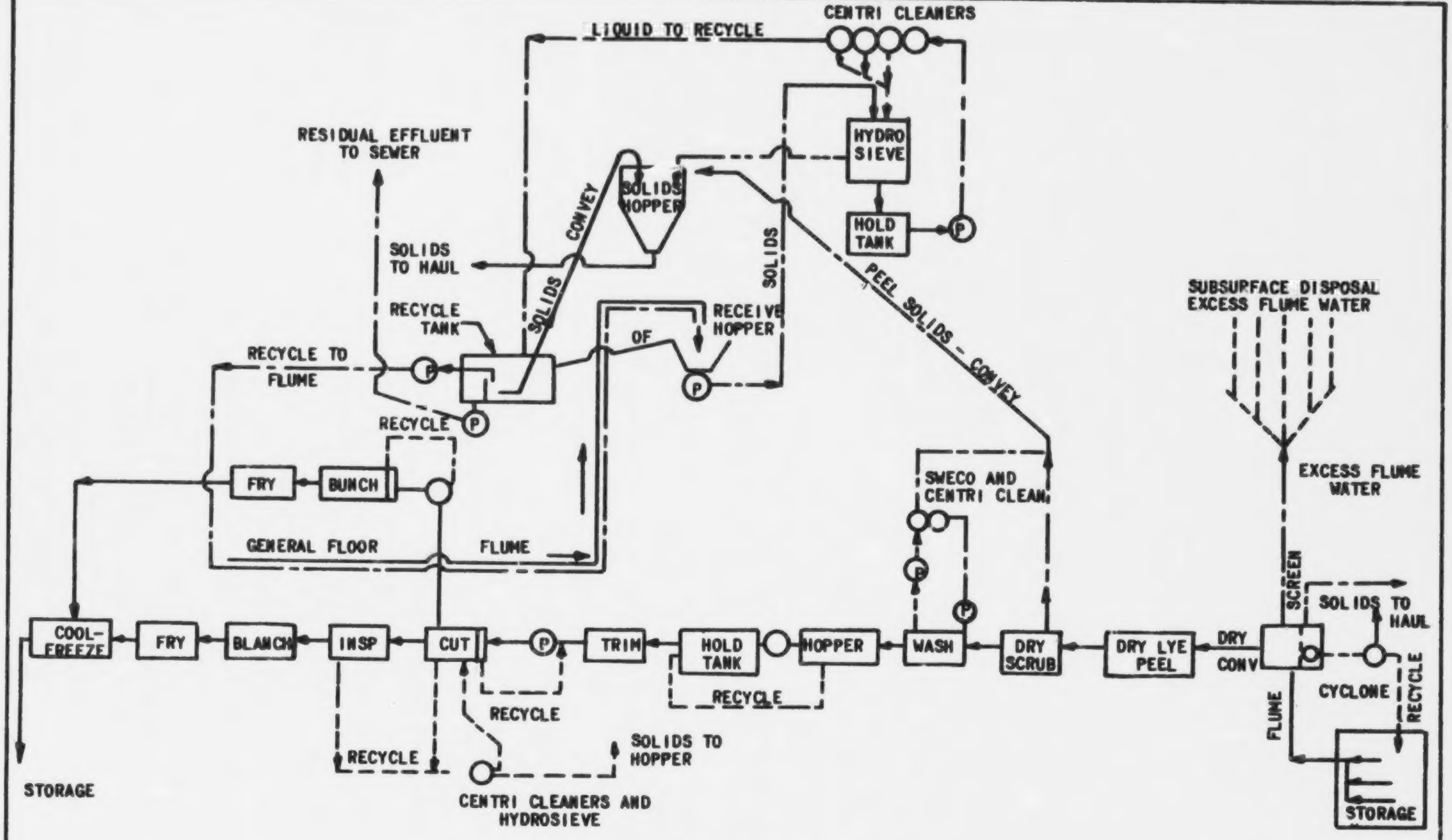
LIQUID TO RECYCLE
CENTRI CLEANERS
RESIDUAL EFFLUENT TO SEWER
HYDRO SIEVE
SOLIDS HOPPER
CONVEY
HOLD TANK
P
SOLIDS TO HAUL
SOLIDS
SOLIDS
RECYCLE TANK
RECEIVE HOPPER
OF
PEEL SOLIDS - CONVEY
SUBSURFACE DISPOSAL EXCESS FLUME WATER
RECYCLE TO FLUME
RECYCLE
FRY
BUNCH
EXCESS FLUME WATER
SWECO AND CENTRI CLEAN
GENERAL FLOOR
FLUME
SCREEN
SOLIDS TO HAUL
COOL-FREEZE
FRY
BLANCH
INSP
CUT
TRIM
HOLD TANK
HOPPER
WASH
DRY SCRUB
DRY LYE PEEL
DRY CONV
CYCLONE
RECYCLE
FLUME
RECYCLE
RECYCLE
RECYCLE
SOLIDS TO HOPPER
STORAGE
CENTRI CLEANERS AND HYDROSIEVE
STORAGE

FIGURE 6
SCHEMATIC PROCESS DIAGRAM
CYR BROTHERS POTATO PROCESSING

will be pumped through a series of centri-cleaners for further reconditioning prior to return to the recycle tank. The concentrated solids from the centri-cleaners will be placed on the hydrosieve.

The overflow from the flume receiving hopper will enter the flume recycle tank, together with the reconditioned liquid component of the solids draw off. Solids which may settle in this receiving and recycle tank will be conveyed by screw conveyors to the solids hopper. A portion of the flow will be pumped to the head of the flume system, while the residual excess will be discharged to the District sewer system. The evaporator cooling waters and defrost waters will also have a recycle system, with any excess entering the flume system.

The Cyr Bros Plant is essentially completed and limited test runs are reported satisfactory. Unfortunately, it is not possible within the time constraints of the report to conduct a full, detailed evaluation of the operation. It would be anticipated that the total water usage and BOD generation of the operation would be well below the industry averages, providing the system functions as planned. However, it is anticipated that a considerable time may be required to de-bug the system and minimize residual waste output. Maximum plant capacity on a potato input basis will probably be between 300 and 400 tons per day, although a more modest production is anticipated in the early years of operation.

General review of the system would suggest that considerable reductions in waste loads may have been achieved, as compared with typical plants. However, several problems may have to be overcome before full potential is realized. The proposal to dispose of excess flume water by subsurface disposal may be difficult to achieve on the long term. This is, of course, related to the residual volume which must be disposed of in this manner, which intturn depends upon the effectiveness of silt removal units. Silt removal is one of the most difficult problems in the industry. It is also noted that the residual wastewater will still carry significant quantities of colloidal silt which may tend to plug the subsurface disposal fields quickly.

The extensive reuse of water by recycling can effect a significant reduction in overall plant water usage, providing reasonable quality control can be maintained. The screening and cyclone-type solids removal units should accomplish a reasonable degree of solids removal, perhaps approaching primary sedimentation. However, the internal water systems will gain in dissolved organic load with each usage. This could create a bacterial or slime problem in the system. In practice, new water will have to be added to the system and wastewater withdrawn at a rate which will maintain acceptable quality. This balance can only be achieved after full operating experience and system adjustment.

The system is dependent upon effective disposal of the collected solids from the collection hopper. Such disposal may be dependent upon achieving a fairly neutral pH and solids consistency. The peel pulp coming from the dry caustic peel unit will have a very high pH. Before this material is acceptable for direct animal feed, and possibly for additional processing into feed, it must be neutralized. This can be accomplished by natural fermentation if time is sufficient. The relatively dry solids from the dry caustic peel operation may be diluted to some extent by solids from the hydrosieve and centrifugal-type solids concentrators. Thus, it is hoped that a reasonable solids consistency can be maintained.

In summary, the Cyr Bros plant represents an attempt to minimize residual wastewater output by utilizing a dry caustic peel procedure, coupled with maximum in-plant water recycle. After adjustment during intial operation, the facility should produce significantly less wastewater load than standard processing plants.

COLBY COOPERATIVE STARCH CO, - Colby Cooperative Starch Co, produces potato starch from low grade potatoes. The residual pulp is processed into an animal feed meal as a by-product. The process is typical of the industry and is illustrated in Figure 7; a generalized process schematic. lant water is a combination of river water, well water, and a modest amount from the municipal supply.

The potatoes are transported from storage to the process area by a water flume system. They are then conveyed to a crusher where the potatoes are pulped and passed to a centrifuge. The centrifuge draws off the protein water and passes the pulp onto a vibrating screen where the starch water is drawn off. The starch water is again centrifuged and screened prior to drying into a starch product. Solids from the centrifuge and screen are returned to the head of the system.

The solids removed from the screening operation are further dewatered, dried and processed into an animal feed product. Essentially all of the plant solids are utilized in this process. The protein water removed during the initial centrifuge operation is further screened and subjected to a second centrifuge operation for additional solids recovery. The solids removed from the protein line are returned for starch recovery. The excess protein water is then discharged either to the river, or the Caribou Utilities District system.

Operation schedules of the Colby plant are quite erratic, which is typical of the industry. Actual operation depends in large part on the availability of culls or low-grade potatoes as a raw material. Total potato intake capacity may approach 450 to 500 tons per 24 hours. However, this theoretical intake capacity is not generally utilized except for short periods. The off-and-on type of operation typical of Maine potato starch plants makes pollution control efforts difficult.

Review of the Colby operation suggests that water usage could be reduced. by recycling a portion of the flume water, perhaps up to 50% recycle. While this is an operating problem, the total water saved could be significant. The primary source of organic pollution is the waste protein water. The unit BOD of the protein may approach 25,000 to 30,000 MG/L. Any significant in-plant organic load reductions must be accomplished in the protein water. Management is currently exploring methods of eliminating the protein water from the waste streams by evaporation to produce a

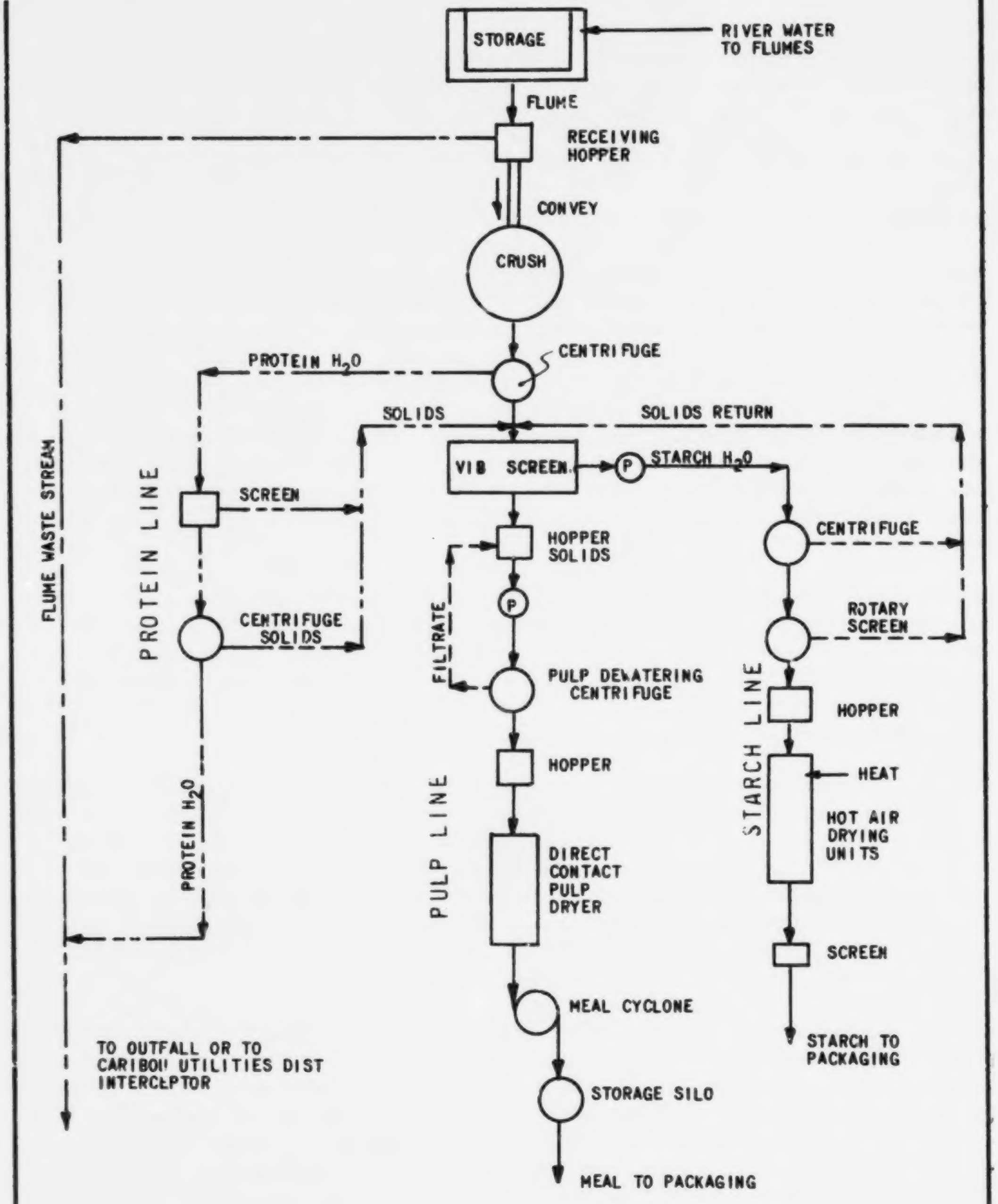
STORAGE
RIVER WATER TO FLUMES
FLUME
RECEIVING HOPPER
CONVEY
CRUSH
CENTRIFUGE
PROTEIN H_2O
SOLIDS
SOLIDS RETURN
VIB SCREEN
P
STARCH H_2O
SCREEN
HOPPER SOLIDS
CENTRIFUGE
FLUME WASTE STREAM
PROTEIN LINE
FILTRATE
P
ROTARY SCREEN
CENTRIFUGE SOLIDS
PULP DEWATERING CENTRIFUGE
STARCH LINE
HOPPER
HOPPER
HEAT
PROTEIN H_2O
PULP LINE
DIRECT CONTACT PULP DRYER
HOT AIR DRYING UNITS
SCREEN
MEAL CYCLONE
TO OUTFALL OR TO CARIBOU UTILITIES DIST INTERCEPTOR
STARCH TO PACKAGING
STORAGE SILO
MEAL TO PACKAGING

FIGURE 7
SCHEMATIC PROCESS DIAGRAM
COLBY COOPERATIVE STARCH COMPANY

recoverable protein product. If successful, such a procedure would greatly reduce the plant's organic pollution load. However, the energy requirements for such a process may prove prohibitive. It may also be possible to remove additional solid materials from the protein water by concentration with cyclone-type equipment. The question of disposal of the concentrated material may or may not be a problem, depending upon availability of a market and haul distances.

Colby management will be faced with difficult decisions concerning its pollution control program. If it can effectively remove the protein water component of the waste stream, its impact on the regional system, and the costs to the company, will be small. If it cannot, the company will be faced with a substantial pollution control cost.

AMERICAN KITCHEN FOODS - The management of American Kitchen Foods, in Caribou, indicated that it has a comprehensive program of in-plant water and waste conservation underway and did not desire review or assistance from the Commission as part of this study. Thus detailed in-plant studies were not accomplished.

In general the American Kitchen Foods plant is typical of the industry, producing frozen french fries and by-products. A water flume system is used for transport from storage to process. A combination of peeling methods is used, with one line reported on a dry caustic peel system. Plant water usage in April of 1971 was measured at about 2.0 MGD. At an approximate potato intake capacity of 440 tons per day, this usage represents about 4500 gallons per ton. This is above the industry average and suggests that considerable reductions may be possible. It is understood that steps are being taken by management to reduce its water usage.

A & P PROCESSING PLANT - The A & P Co processing plant in Fort Fairfield produces frozen french fires and frozen by-products during the potato processing season, and processes frozen peas during a period in the summer. A generalized schematic process diagram of the A & P operation is shown in Figure 8. Again this diagram reflects conditions during the gaging and sampling period in the spring of 1971.

Potatoes are transferred dry from storage to the processing area. The A & P plant is the only one in the County which does not use a flume transport system. The potatoes are first washed for removal of silt and rocks which may be in the storage boxes. The washed potatoes then enter the peeling process, which consists of a rather unique combination of lye-peel and steam-peel system. The relative use of each component of the system varies with the difficulties encountered in peeling the potatoes to be processed.

Subsequent to peeling, the potatoes are cut, trimmed, blanched and fried in normal industry procedure. Typical of the industry, slivers, nubbings, etc, are utilized in a frozen by-products line. Process water is taken from Fort Fairfield's public water supply. As indicated in the diagram,

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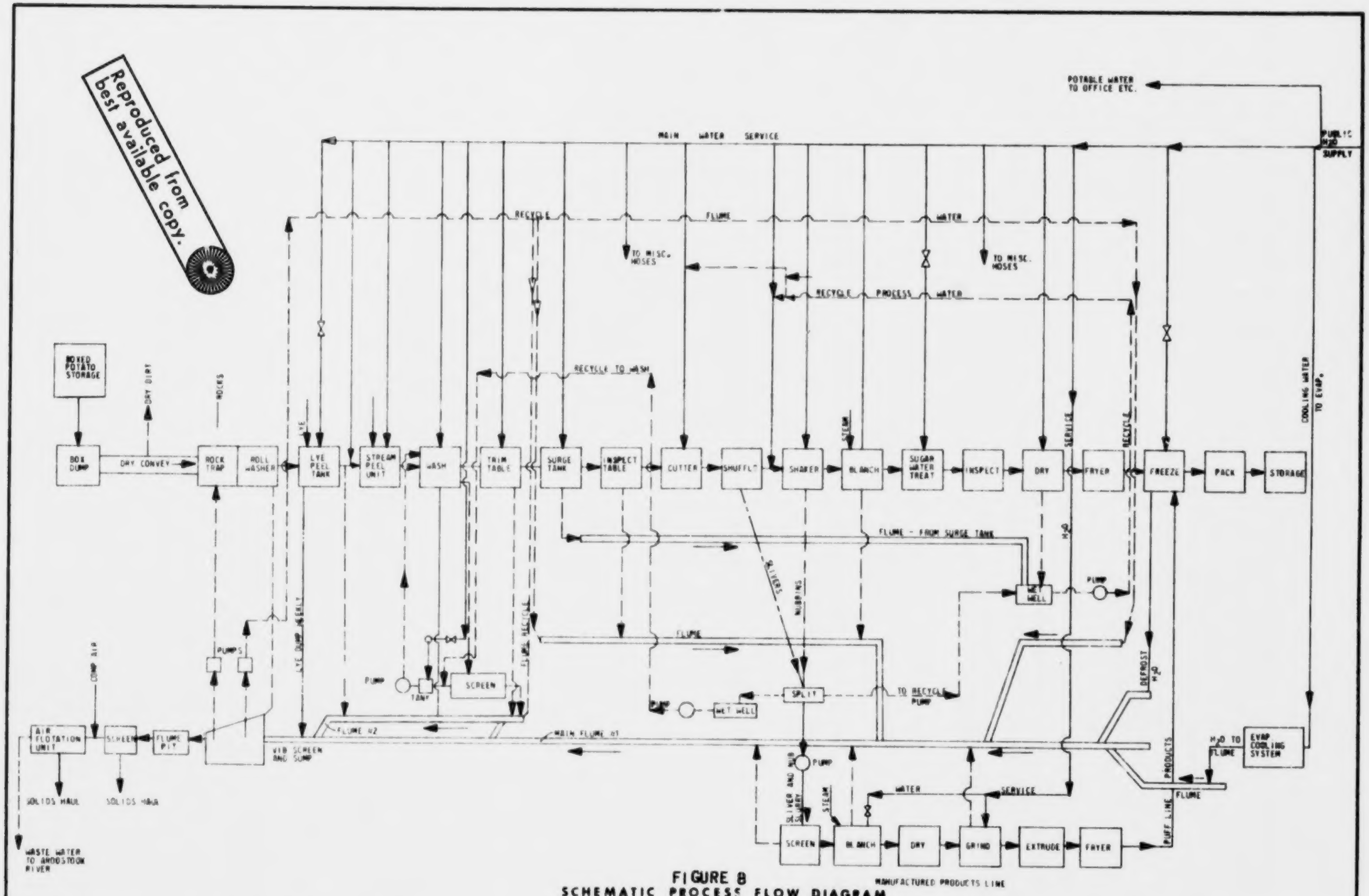
POTABLE WATER TO OFFICE ETC.
PUBLIC H2O SUPPLY
MAIN WATER SERVICE
RECYCLE
FLUME
WATER
TO MISC. HOSES
TO MISC. HOSES
RECYCLE PROCESS WATER
RECYCLE TO WASH
COOLING WATER TO EVAP.
BOXED POTATO STORAGE
BOX DUMP
DRY DIRT
DRY CONVEY
ROCKS
ROCK TRAP
ROLL WASHER
LYE
LYE PEEL TANK
STREAM PEEL UNIT
WASH
TRIM TABLE
SURGE TANK
INSPECT TABLE
CUTTER
SHUFFLE
SHAKER
STEAM
BLANCH
SUGAR WATER TREAT
INSPECT
DRY
SERVICE
FRYER
RECYCLE
FREEZE
PACK
STORAGE
FLUME - FROM SURGE TANK
H2O
SLIVERS
NUBBINS
WET WELL
PUMP
FLUME
LYE DUMP WEEKLY
FLUME RECYCLE
PUMPS
COMP AIR
PUMP
TANK
SCREEN
SPLIT
TO RECYCLE PUMP
PUMP
WET WELL
DEFROST H2O
FLUME #2
MAIN FLUME #1
VIB SCREEN AND SUMP
FLUME PIT
SCREEN
AIR FLOTATION UNIT
SOLIDS HAUL
SOLIDS HAUL
WASTE WATER TO AROOSTOOK RIVER
PUMP
SLIVER AND NUB SLURRY
STEAM
WATER
SERVICE
SCREEN
BLANCH
DRY
GRIND
EXTRUDE
FRYER
PUFF LINE
PRODUCTS
FLUME
H2O TO FLUME
EVAP COOLING SYSTEM
MANUFACTURED PRODUCTS LINE

FIGURE 8
SCHEMATIC PROCESS FLOW DIAGRAM
A & P PROCESSING PLANT

recoverable protein product. If successful, such a procedure would greatly reduce the plant's organic pollution load. However, the energy requirements for such a process may prove prohibitive. It may also be possible to remove additional solid materials from the protein water by concentration with cyclone-type equipment. The question of disposal of the concentrated material may or may not be a problem, depending upon availability of a market and haul distances.

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In general the American Kitchen Foods plant is typical of the industry, producing frozen french fries and by-products. A water flume system is used for transport from storage to process. A combination of peeling methods is used, with one line reported on a dry caustic peel system. Plant water usage in April of 1971 was measured at about 2.0 MGD. At an approximate potato intake capacity of 440 tons per day, this usage represents about 4500 gallons per ton. This is above the industry average and suggests that considerable reductions may be possible. It is understood that steps are being taken by management to reduce its water usage.

A & P PROCESSING PLANT - The A & P Co processing pl ant in Fort Fairfield produces frozen french fires and frozen by-products during the potato processing season, and processes frozen peas during a period in the summer. A generalized schematic process diagram of the A & P operation is shown in Figure 8. Again this diagram reflects conditions during the gaging and sampling period in the spring of 1971.

Potatoes are transferred dry from storage to the processing area. The A & P plant is the only one in the County which does not use a flume transport system. The potatoes are first washed for removal of silt and rocks which may be in the storage boxes. The washed potatoes then enter the peeling process, which consists of a rather unique combination of lye-peel and steam-peel system. The relative use of each component of the system varies with the difficulties encountered in peeling the potatoes to be processed.

Subsequent to peeling, the potatoes are cut, trimmed, blanched and fried in normal industry procedure. Typical of the industry, slivers, nubbings, etc, are utilized in a frozen by-products line. Process water is taken from Fort Fairfield's public water supply. As indicated in the diagram, the plant makes reasonable use of internal recycling. The overall system generated about 0.8 MGD of wastewater during the testing period in the spring of 1971. With raw potato intake approximately 215 tons per day, unit water usage is just under 4000 gallons per ton. This is only slightly below the industry average and suggests that some reductions may be possible.

The waste stream combines the wash and process waters, and passes through screening and air flotation units for solids removal prior to discharge to the Aroostook River. The screenings are truck handled for processing into animal feed, while the air-floated solids are disposed of at the municipal dump. The effectiveness of these treatment units is discussed in a later section of this report.

A review of the A & P operation in the spring of 1971 suggested several procedures for consideration by management for reducing waste loads, prior to external treatment. The suggestions for in-plant adjustments are briefly described as follows:

1. Recycle and reuse water from the evaporation system.
2. Install automatic valves on general use hoses and undertake in-plant education on their use.
3. Dry haul the solids from the wash water recycle screen.
4. Separate initial wash water from the other process water, and install mud removal facilities prior to discharge to the effluent pipe.
5. Install dry caustic peel procedures to replace existing peeling equipment.
6. Utilize cyclone type solids removal units on certain waste streams, to remove concentrated protein water for salvage.

Item 1 above constitutes the largest single potential reduction in plant water usage. Currently, the water used for cooling the evaporators is used once and discharged into the head of the flume system. This volume varies with the outside air temperature and the demands of the evaporating system, but can reach an estimated 180,000 to 200,000 gallons per day. This water is essentially unpolluted after passing through the evaporator and it should be possible to reintroduce this water into the plant flow. Makeup water from the district mains can be allowed to enter the process water system directly with a backflow preventer to safeguard the potable water system.

Item 2 is primarily a routine housekeeping operation to minimize miscellaneous water use. It is recognized that plant operating personnel may dislike such devices, and may attempt to circumvent their use. However, in-plant water use can be curtailed by this method, when coupled with an education program for personnel in the significance of water conservation.

Item 3 above will not reduce plant flow, but effect have some reduction in plant BOD loading by removing solids from the waste stream as rapidly as possible. This will minimize the leaching of organics from the solids into the solution.

Item 4 above also will not significantly reduce plant flow. However, it may ease maintenance problems in the existing sewer and at treatment facilities, by removing much of the mud before it can settle out in the sewer lines. Handling mud from a potato washing operation is difficult at best, but it can probably be handled in a mud pit or sump more easily than it can be removed from a pipe line.

Item 5 above considers replacement of the existing peeling equipment with a dry caustic peel line with semi-dry haul of peel wastes. Again comments on such a system are similar to those discussed in the prior Taterstate discussion.

Item 6, concerning the use of cyclone-type solids concentrations for starch recovery, is similar to that discussed above for the Taterstate and Potato Service Inc, plants.

The potential for residual waste load reduction through implementation of the above alterations is presented in a later section of this report. Again, prior to final design of pollution abatement facilities, management must determine the applicability of each as related to their product and quality control procedures.

VAHLSING, INC - The Vahlsing, Inc potato processing plant in Easton was not studied in detail concerning its internal operations. The plant is generally typical of the industry, producing frozen french fries and byproducts. Water fluming is used together with the lye peeling process. As the plant is relatively modern, a less than average water usage would be anticipated. The prior comments on internal waste load reduction, particularly the dry caustic peel system, are equally applicable to the Vahlsing plant.

MAINE SUGAR INDUSTRIES - The Maine Sugar Industries plant in Easton was constructed in the late 1960's to process the newly acquired State sugar beet allotment. Unfortunately, the growing of beets in Aroostook County has proven difficult. This, combined with financial difficulties inherent in a new venture, has prevented full operation of the facility. To date only a limited volume of beets has been processed, and no production occurred during the course of this study. The future of the operation is uncertain.

The plant is capable of processing refined sugar from either beets or cane. It is a modern plant designed by Braunschweigische Maschinenbaunstatt of Braunschweig, West Germany. The general process description and schematic process diagram shown in Figure 9 are taken from the Camp Dresser and McKee 1970[33] report to the Town of Easton.

The beets are transported from storage to process by a water flume system. The fluming system is theoretically closed, although solids buildup requires partial makeup and wasting. During the winter the beets are warmed by this fluming process. The beets are then cut into cossettes, and sugar is extracted in a continuous diffusion tower in the form of raw juice which is 10% to 13% pure sugar. The spent cossettes are pressed, dried and used for animal feed.

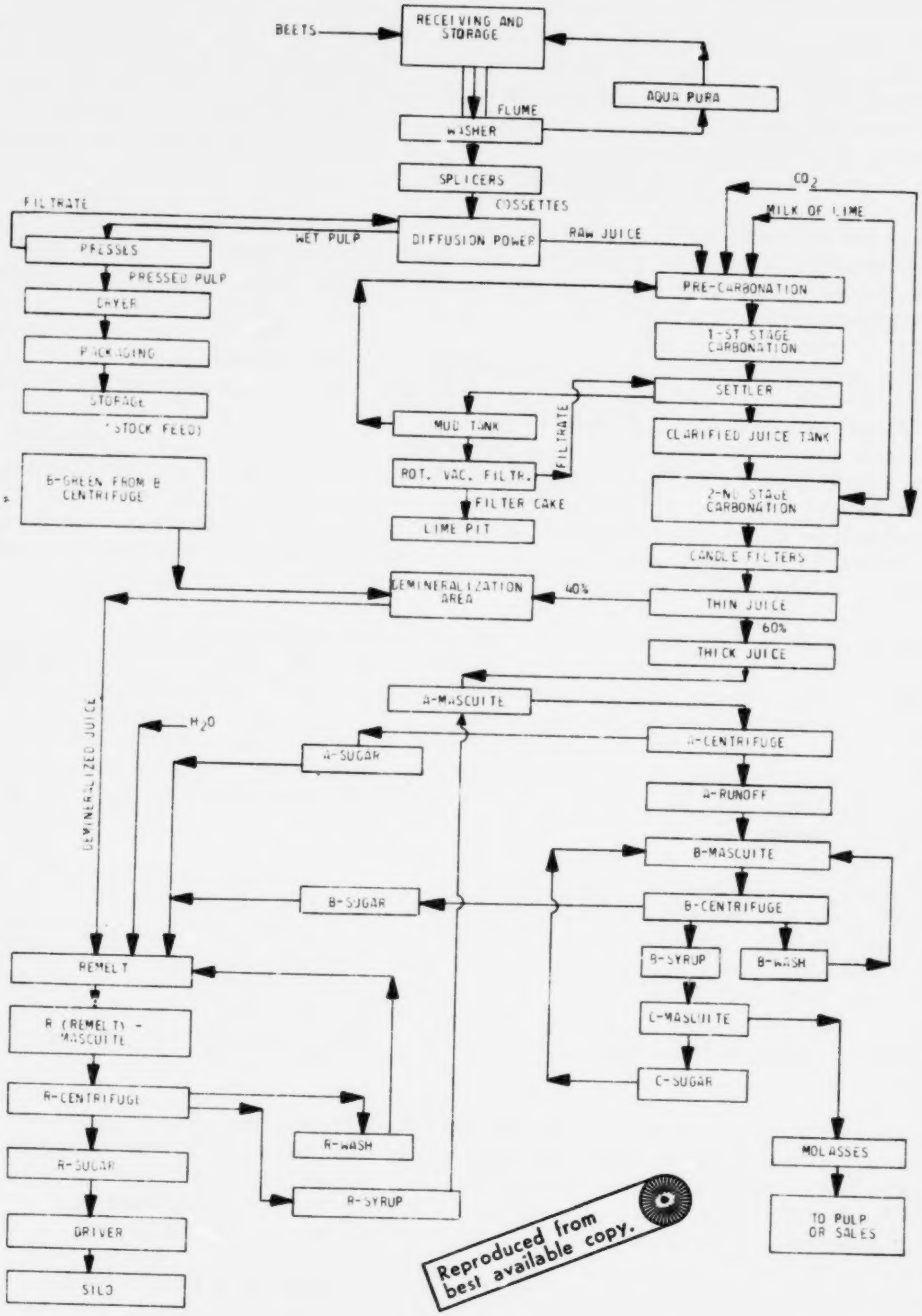
BEETS
RECEIVING AND STORAGE
AQUA PURA
FLUME
WASHER
SPLICERS
COSSETTES
CO2
FILTRATE
MILK OF LIME
WET PULP
DIFFUSION POWER
RAW JUICE
PRESSES
PRESSED PULP
PRE-CARBONATION
DRYER
1-ST STAGE CARBONATION
PACKAGING
SETTLER
STORAGE
(STOCK FEED)
MUD TANK
FILTRATE
CLARIFIED JUICE TANK
B-GREEN FROM B CENTRIFUGE
ROT. VAC. FILTR.
2-ND STAGE CARBONATION
FILTER CAKE
LIME PIT
CANDLE FILTERS
DEMINERALIZATION AREA
40%
THIN JUICE
60%
THICK JUICE
A-MASCUITE
DEMINERALIZED JUICE
H2O
A-CENTRIFUGE
A-SUGAR
A-RUNOFF
B-MASCUITE
B-SUGAR
B-CENTRIFUGE
B-SYRUP
B-WASH
REMELT
C-MASCUITE
R (REMELT) - MASCUITE
C-SUGAR
R-CENTRIFUGE
R-WASH
MOLASSES
R-SUGAR
R-SYRUP
TO PULP OR SALES
DRIVER
SILO

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FIGURE 9
SCHEMATIC PROCESS FLOW DIAGRAM
MAINE SUGAR INDUSTRIES

The raw juice is purified in two carbonation stages, with the addition of carbon dioxide and milk of lime. The precipitate is removed by settling and candle filtration. Two stages are used in this operation. The solids from both stages are vacuum filtered and air-ejected to the mud-lime pit.

Sixty percent of the juice is thickened and subjected to three repeated cycles of evaporation and centrifuging to produce declining quality A, B and C sugars. The C sugar is either mixed with the pulp for feed or sold as molasses. The remaining 40 percent is demineralized and subjected to a remelt process, which is mixed with the low grade streams from the A and B centrifuges, evaporated and centrifuged. The R sugar thus produced, is dried and stored for packaging.

The basic process is more characteristic of the straight house operation than the Steffens operation, and makes extensive use of water recycling. This eliminates many of the typical pollution sources in the United States sugar beet industry. Plant production capacity is reported at 4000 tons of beets per day. Operation would be on a 24 hour per day basis during the operating season, which could run about 100 days in the late fall and winter. Plant water usage was estimated at about 2000 GPM in the Camp Dresser and McKee report[33]. Operation of the plant to date has not allowed detailed flow or organic load analysis.

WASTEWATER CHARACTERIZATION

Flow gagings, sampling, and analyses were conducted at all processing plants, except Vahlsing, Inc, and Maine Sugar Industries at Easton, and the Cyr Bros plant in Caribou. The program was conducted in April, and early May, 1971. The analyses conducted during the spring of the year represent the maximum organic loads to be anticipated; due to the difficulties in peeling old potatoes and the increase in spoiled or partially rotted potatoes which may enter the system. It should also be noted that the picking season in the fall of 1970, was one of the wettest in years, and large quantities of mud entered the storage and fluming systems. In general, the recorded values should represent maximum loadings for production being accomplished by the industry.

All laboratory analyses performed in conjunction with the characterization of the wastewaters, from the potato processing plants, were made in conformance with the requirements of Standard Methods for the Examination of Water and Wastewater, Thirteenth Edition[35] and FWPCA Methods for Chemical Analysis of Water and Wastes, November, 1969.[36] Analyses were performed within 24 hours of the completion of the sampling period, with the exception of the nitrogen and phosphorous examinations. Samples for these analyses were preserved in accordance with the references listed above.

TATERSTATE FROZEN FOODS - Samples were composited on an 8 hour basis for the 24 hour periods of 7 am, April 6, 1971 to 7 am, April 7, 1971; and from 11 pm, April 7, 1971 to 11 pm, April 8, 1971. This constitutes a

48 hour testing period. An additional limited test was also run on April 28, 1971. A summary of the test data on the raw waste loads (after screening) is presented in Table 1.

TABLE 1

WASTE CHARACTERISTIC SUMMARY

TATERSTATE - RAW WASTES AFTER SCREENING

Date	Flow MGD	Ave BOD MG/L	Ave COD MG/L	Ave Total Solids MG/L	Ave Fixed TS MG/L	Ave SS MG/L
4-6 to 4-7 1971	0.986	2977	4545	4620	715	2522
4-7 to 4-8 1971	0.972	3568	5265	5100	931	3050
4-28-71 7 hrs. only	0.320	3200	--	—	--	—

Date	Ave Fixed SS MG/L	Ave Sett Sol ML/L	Ave Alka MG/L($CaCO_3$)	Chlorides MG/L	pH	Temp °F
4-6 to 4-7 1971	426	48	83	--	6.0	77
4-7 to 4-8 1971	564	42	89	51	6.1	77

The waste characteristics presented in Table 1 can be converted to total daily waste loads, and to waste loads per ton of raw potato input. This data form is necessary for preliminary design, and allows comparison with generally available literature values. Table 2 presents measured total waste load generation at the Taterstate plant.

The Taterstate plant is equipped with a sedimentation-flotation device for solids removal, prior to wastewater discharge. Tests were conducted on the sedimentation-flotation unit effluent, concurrent with the influent testing described above. A summary of the test data on the waste loads, after sedimentation-flotation, is presented in Table 3.

The waste characteristics presented in Table 3 have also been converted to total waste loads, and unit waste loads per ton of potato after sedimentation-flotation. This total load data is presented in Table 4.

TABLE 2

WASTE LOAD SUMMARY

TATERSTATE - RAW WASTES AFTER SCREENING

Date	Unit	Total Amount	Unit Load/Ton Potato
4-6 to 4-7 1971	BOD	24,400 lbs/d	72.0 lbs/t
	COD	37,300 lbs/d	110.0 lbs/t
	SS	20,800 lbs/d	61.4 lbs/t
	Flow	0.986 mgd	2900 gal/t
4-7 to 4-8 1971	BOD	29,600 lbs/d	98.4 lbs/t
	COD	42,600 lbs/d	141.2 lbs/t
	SS	24,700 lbs/d	82.0 lbs/t
	Flow	0.972 mgd	3230 gal/t
4-28-71	BOD	21,050 lbs/d±	60.5 lbs/t±
	Flow	1.097 mgd±	3138 gal/t±

TABLE 3

WASTE CHARACTERISTIC SUMMARY

TATERSTATE- WASTES AFTER SEDIMENTATION-FLOTATION

Date	Flow MGD	Ave BOD MG/L	Ave COD MG/L	Ave Total Solids MG/L	Ave Fixed TS MG/L	Ave SS MG/L
4-6 to 4-7 1971	0.986	2313	4043	4168	771	1941
4-7 to 4-8 1971	0.972	3056	4560	4357	849	2107
4-28-71 7 hrs. only	0.320	2100	--	--	--	--

Date	Ave Fixed SS MG/L	Ave Sett Sol ML/L	Ave Alka MG/L($CaCO_3$)	Chlorides MG/L	pH	Temp °F
4-6 to 4-7 1971	348	27	65	--	5.6	77
4-7 to 4-8 1971	455	33	76	55	5.6	--

Date	Total P MG/L	Soluble P MG/L	Ortho P MG/L	Ammonia N MG/L	Total N MG/L
4-6 to 4-7 1971	40.4	37.0	29.4	0.005	0.085
4-7 to 4-8 1971	40.2	35.4	26.4	0.007	0.107

TABLE 4

WASTE LOAD SUMMARY

TATERSTATE - WASTES AFTER SEDIMENTATION-FLOTATION

Date	Unit	Total Amount	Unit Load/Ton Potato
4-6 to 4-7 1971	BOD	19,000 lbs/d	56.1 lbs/t
	COD	33,200 lbs/d	97.8 lbs/t
	SS	16,050 lbs/d	47.4 lbs/t
	Flow	0.986 mgd	2900 gal/t
4-7 to 4-8 1971	BOD	24,700 lbs/d	82 lbs/t
	COD	37,000 lbs/d	123 lbs/t
	SS	17,050 lbs/d	56.6 lbs/t
	Flow	0.972 mgd	3230 gal/t
4-28-71 7 hrs.	BOD	19,200 lbs/d	55 lbs/t
	Flow	1.097 mgd	3138 gal/t

The wastewaters from the Taterstate plant are generally typical of plants processing potatoes into french fries and potato by-products, utilizing the steam peel process. Table 5 shows a comparison of average Taterstate loads per ton of potatoes processed to standard literature values.

TABLE 5

COMPARISON OF TATERSTATE WASTES WITH LITERATURE
(Raw Wastes - After Screening)

Item	Taterstate	Lit Low	Lit Ave	Lit High
BOD	70 lbs/t±	22 lbs/t	51 lbs/t	90 lbs/t
SS	65 lbs/t±	25 lbs/t	61 lbs/t	114 lbs/t
Flow	3000 gal/t±	2310 gal/t	4210 gal/t	7000 gal/t

The data presented in Table 5 are representative discharges from the Taterstate plant. It is noted that unit waste generation on April 7-8, 1971, was considerably higher than that obtained on April 6-7, 1971. However, on April 8, 1971, the plant was preparing to shut down for the long Easter weekend. This increased the general clean up flows, tank dumps, etc, while actual production tailed off. This resulted in the very high unit waste production and does not represent normal conditions. An additional run to confirm this observation was conducted on April 28, 1971.

Analysis of Table 5 shows that the waste volume generated at the Taterstate plant is well below the industry average and indicates relatively close in-plant control of water usage. The suspended solids value appears to be about the industry average. However, the unit BOD production is somewhat above the published industry average. This value reflects spring run conditions, when the stored potatoes tend to be of lower quality, and peeling is more difficult. It is anticipated that fall conditions would indicate a lower unit BOD production. The remaining data presented, appears typical of a processing plant utilizing a steam peel operation.

POTATO SERVICE, INC - Samples were composited on an 8 hour basis for a 48 hour period on April 27 to 29, 1971. Samples were taken at 5 locations; namely, on the flume wastewater line before and after sedimentation, on the process wastewater line before and after sedimentation, and on the final effluent before it enters the river. Table 6 presents the flume line data; Table 7 presents the process line data; and Table 8 presents the final effluent data.

TABLE 6

WASTE CHARACTERISTIC SUMMARY

POTATO SERVICE, INC - FLUME LINE

Before and After Sedimentation

Date	Flow MGD	Ave BOD MG/L	Ave COD MG/L	Ave Total Sol MG/L	Ave Fixed Total Sol	Ave SS MG/L
4-27 to 4-29 1971	0.5	522	1263	3356	2026	2236
		254	577	1863	1159	533
4-28 to 4-29 1971	0.5	340	952	3084	2027	1519
		182	433	1789	1224	286

Date	Ave Fixed SS MG/L	Ave Sett Sol ML/L	Ave Alka MG/L($CaCO_2$)	Chlorides MG/L	pH	Temp °F
4-27 to 4-29 1971	1369	15	290	--	10.0	--
	323	0.7	283	464		79
4-28 to 4-29 1971	937	13	260	--	9.8	--
	286	1.5	220	398		74

NOTE: Higher Values = Raw Load
Lower Values = After Sedimentation

TABLE 7

WASTE CHARACTERISTIC SUMMARY

POTATO SERVICE, INC - PROCESS LINE

Before and After Sedimentation

Date	Flow MG/D	Ave BOD MG/L	Ave COD MG/L	Ave Total Sol MG/L	Ave Fixed Tot Sol MG/L	Ave SS MG/L
4-27 to 4-28 1971	2.6	2893	5460	6340	1419	2828
		2188	3745	4539	1459	391
4-28 to 4-29 1971	2.6	2793	5592	5519	1491	2418
		2123	3453	4602	1541	327

Date	Ave Fixed SS MG/L	Ave Sett Sol ML/L	Ave Alka MG/L(Ca CO_3)	Chlorides MG/L	pH	Temp °F
4-27 to 4-28 1971	310	68	1003	--	11.2	--
	107	0.1	739	69	10.8	82
4-28 to 4-29 1971	332	70	1080	--	11.5	—
	108	0.2	670	89	11.0	82

NOTE: Higher Values = Wastes after Screening
Lower Values = Wastes After Sedimentation

TABLE 8

WASTE CHARACTERISTIC SUMMARY

POTATO SERVICE, INC - POND EFFLUENT

Date	Flow MGD	Ave BOD MG/L	Ave COD MG/L	Ave Total Sol MG/L	Ave Fixed Tot Sol MG/L	SS MG/L
4-27 to 4-28 1971	3.6	669	1593	2815	1314	843
4-27 to 4-28 1971	3.6	812	1778	2836	1338	1097

Date	Ave Fixed SS MG/L	Ave Sett Sol ML/L	Ave Alka MG/L($CaCO_3$)	Chlorides MG/L	pH	Temp °F
4-27 to 4-28 1971	190	0.6	975	--	7.4	54
4-28 to 4-29 1971	361	0.5	1080	--	7.4	54

TABLE 8
(CONTINUED)

Date	Phos Ortho	MG/L Tot	Ammonia MG/L-N	Organic N MG/L-N	NO_3 MG/L-N	NO_2 MG/L-N	Total N MG/L N
4-27 to 4-28 1971	19.4	34.6	0.017	0.04	0.12	0.00	0.141
4-28 to 4-29 1971	19.2	33.0	0.009	0.04	0.15	0.00	0.199

NOTE: Pond has 10 to 12 days detention - effluent cannot be compared directly with plant effluent to determine efficiency.

-Processing plant had been out of production for over a week about 10 days prior to testing.

-Effluent showed an increase in strength over the testing period. Based on earlier PSI data, an effluent BOD of about 1000 MG/L may be expected.

Data from the above tables can be converted to total loads, and to unit loads per ton of potatoes. This data format allows comparison with literature values, and permits assembly of design loading for system design. The total plant load and unit load per ton of potatoes is shown in Table 9.

Analysis of Table 9 shows the total BOD generated to be about 63,000 lbs/day. At production levels on the days of testing, this represents a generation of about 62 lbs BOD per ton of potato input. This is about 20% above the industry average of 51 lbs/t, as reported in literature. Suspended solids flow generation, and the other reported data appears to be in line with industry averages, for a plant using the lye peel process.

COLBY COOPERATIVE STARCH - Gaging and sampling was conducted on three different days at the Colby plant. Each run was about 4 to 5 hours, corresponding generally to plant production. Samples were taken on the in-plant protein water line, and on the overall plant outfall. The flow data obtained is approximate only, as foaming caused by the protein water created difficulties in securing accurate weir measurements. The data obtained during the testing program is shown on Table 10.

Analysis of Table 10 indicates that a significant organic waste load is generated when the Colby plant is operating. The flow is about 1000 gallons per ton of potato input, which is mostly used for fluming. As noted in the previous section, recycling of the flume water could reduce the total waste flow. The major organic component of the waste is created by the protein water. The organic strength of the wastewater

TABLE 9

LOAD SUMMARY - POTATO SERVICE, INC

Location	4-27 to 4-28, 1971			4-28 to 4-29, 1971		
	BOD	COD	SS	BOD	COD	SS
Inf Flume	2,170 lbs	5,260 lbs	9,300 lbs	1,415 lbs	3,960 lbs	6,320 lbs
Tank	2.3 lbs/t	5.5 lbs/t	9.7 lbs/t	1.3 lbs/t	3.7 lbs/t	5.9 lbs/t
Eff Flume	1,060 lbs	2,400 lbs	2,220 lbs	758 lbs	1,809 lbs	1,190 lbs
Tank	1.1 lbs/t	2.5 lbs/t	2.3 lbs/t	0.7 lbs/t	1.7 lbs/t	1.1 lbs/t
Inf to Process	62,600 lbs	117,500 lbs	61,200 lbs	60,600 lbs	121,100 lbs	52,500 lbs
Tank	65.3 lbs/t	1122.5 lbs/t	63.7 lbs/t	56.8 lbs/t	113.5 lbs/t	49.1 lbs/t
Eff to Process	47,400 lbs	81,200 lbs	8,500 lbs	46,000 lbs	74,700 lbs	7,100 lbs
Tank	49.5 lbs/t	84.6 lbs/t	8.9 lbs/t	43.0 lbs/t	70.0 lbs/t	6.7 lbs/t
Inf to Ponds	48,460 lbs	83,600 lbs	10,720 lbs	46,750 lbs	76,509 lbs	8,290 lbs
	50.5 lbs/t	87.2 lbs/t	11.2 lbs/t	43.7 lbs/t	70.2 lbs/t	7.8 lbs/t
Eff to Ponds	17,300 lbs	41,200 lbs	21,700 lbs	21,000 lbs	45,900 lbs	28,300 lbs
	18.0 lbs/t	43.9 lbs/t	22.6 lbs/t	19.6 lbs/t	43.0 lbs/t	26.5 lbs/t

TABLE 10

WASTE CHARACTERISTIC SUMMARY

COLBY CO-OP STARCH PLANT

Raw Waste Water

Date	Flow GPM	Hrs of Run	Total Flow MG	Ave BOD MG/L	Ave COD MG/L	Ave Tot Sol MG/L
4-21-71 Plant Eff.	370±	5	0.111	5,700	8,950	9,035
4-21-71 Protein H_2O Portion	70±	4-1/2	0.019	28,900	45,200	20,300

Date	Ave Fixed Tot Sol MG/L	Ave SS MG/L	Ave Fixed SS MG/L	Sett Sol ML/L	pH	Temp °F
4-21-71 Plant Eff	1830	3355	710	48	6.8	--
4-21-71 Protein H_2O Portion	7660	6680	825	--	6.1	--

Date	Flow GPM	Hrs Of Run	Total PO_4 as P MG/L	Ortho PO_4 as P MG/L	Org Nit as N MG/L	NO_3(N) MG/L
4-21-71 Plant Eff	570	5	48.8	26.8	.04	.42
4-21-71 Protein H_2O Portion	70±	4-1/2	296	275	--	8.0

Date	NO_2(N) MG/L	NH_3(N) MG/L	Total N MG/L
4-21-71 Plant Eff	0	.12	.58
4-21-71 Protein H_2O Portion	0	.09	--

Date	Flow GPM	Hrs of Run	Total Flow MG	Ave BOD MG/L	Ave COD MG/L	Ave Total Sol MG/L
6-2-71 Plant Eff.	370±	4	.08	3500	8700	7800
6-2-71 Protein H_2O Portion	70±	4	.017	14,900	37,600	28,500

TABLE 10
(CONTINUED)

Date	Ave Fixed Tot Sol MG/L	Ave SS MG/L	Ave Fixed SS MG/L	Sett Sol ML/L	pH	Temp °F
6-2-71 Plant Eff	2100	3130	840	--	--	58
6-2-71 Protein H_2O Portion	7500	4760	840	--	--	--

Date	Flow GPM	Hrs of Run	Total Flow MG	Ave BOD MG/L	Ave COD MG/L	Ave Total Sol MG/L
6-3-71 Plant Eff	370±	4-1/2	0.09	3580	7840	6750
6-3-71 Protein H_2O Portion	70±	4-1/2	0.019	17100	35300	27700

Date	Ave Fixed Tot Sol MG/L	Ave SS MG/L	Ave Fixed SS MG/L	Sett Sol ML/L	pH	Temp °F
6-3-71 Plant Eff	1850	1900	400	--	--	58
6-3-71 Protein H_2O Portion	7300	3360	460	--	--	--

was less during the June sampling than during April. No direct explanation for this variation is apparent, although it is likely that the total organic output may vary considerably depending on the quality of input potato. Total organic load generation of the plant probably ranges from 15,000 to 20,000 lbs/d BOD, if a full production day was achieved. It must be noted, however, that production is erratic and depends on the raw material supply. Full production days are not the norm.

AMERICAN KITCHEN FOODS - Samples were composited on an 8 hour basis for the 48 hour period April 20 to April 23, 1971. Samples were taken at three locations; namely, the American Kitchen Foods plant effluent, after screening; the influent to the treatment plant of the Caribou Utilities District; and the effluent from the treatment plant during the periods when it was operating. At the time of testing, American Kitchen Foods was the only significant industry discharging to the Caribou system. Table 11 presents the American Kitchen Foods effluent data. Table 12 presents both the influent and effluent data at the Caribou treatment plant.

It should be noted that the effluent data from Table 12 represents only the period 8:30 am to 4 pm. During the remaining part of the testing, on that date, the sedimentation units were being by-passed. This bypassing was required primarily because of the plant's inability to handle the solids load generated.

TABLE 11

WASTE CHARACTERISTIC SUMMARY

AMERICAN KITCHEN FOODS PLANT

Wastes After Screening

Date	Flow MGD	Ave BOD MG/L	Ave COD MG/L	Ave Total Sol MG/L	Ave Fixed Tot Sol MG/L	Ave SS MG/L
4-20 to 4-21 1971	2.0±	1850	2553	3384	570	1511
4-22 to 4-23 1971	2.0±	1810	2483	2226	454	1833

Date	Ave Fixed SS MG/L	Ave Sett Sol ML/L	Ave Alka MG/L($CaCO_3$)	Chlorides MG/L	pH	Temp °F
4-20 to 4-21 1971	267	33	54	--	6.5	72
4-22 to 4-23 1971	131	30	151	36	7.1	72

Date	Phos-MG/L Ortho	Phos-MG/L Tot	Ammonia MG/L-N	Organic MG/L-N	NO_3 MG/L-N	NO_2 MG/L-N	Total N MG/L N
4-20 to 4-21 1971	16.6	28.2	0.006	0.020	0.35	0.00	0.376
4-22 to 4-23 1971	12.2	23.4	--	--	--	--	--

TABLE 12

WASTE CHARACTERIZATION SUMMARY

CARIBOU TREATMENT PLANT

Influent & Effluent

Date	Flow MGD		Ave BOD MG/L	Ave COD MG/L	Ave Total Sol MG/L	Ave Fixed Tot Sol MG/L	Ave SS MG/L
4-20 to 4-21	4.0±	Inf	647	1033	1427	371	556
1971		Eff	390	610	907	352	184
4-22 to 4-23	3.6±		678	925	1337	360	475
1971			435	640	982	357	164

Date	Ave Fixed SS MG/L	Ave Sett Sol ML/L	Ave Alka MG/L($CaCO_3$)	Chlorides MG/L	pH	Temp °F
4-20 to 4-21	101	11	99	--	6.8	--
1971	69	0.6	120		--	--
4-22 to 4-23	65	11	117	55	7.7	--
1971	50	1	104		7.0	--

TABLE 12
(CONTINUED)

	Phos MG/L		Ammonia	Organic	NO_3	NO_2	Total N
Date	Ortho	Tot	MG/L-N	MG/L-N	MG/L-N	MG/L-N	MG/L
4-20 to 4-21	5.0	8.1	--	0.019	1.20	0.022	--
1971	6.2	11.4	0.004	0.04	0.81	0.000	0.854
4-22 to 4-23	--	--	0.002	0.033	1.19	0.00	1.225
1971	14.8	22.0	0.006	0.036	1.13	0.00	1.172

The data compiled at American Kitchen Foods, and the Caribou treatment plant have been processed to yield total daily load figures, and load generated per ton of potatoes. It is noted that the unit loads are based on approximate potato intake data only, as actual production data was not available. Table 13 shows the effluent data from the American Kitchen Foods plant, and Table 14 shows similar data for the Caribou treatment plant influent. Due to the limited use of the treatment plant, data on the overall effluent is not meaningful.

TABLE 13

TOTAL LOAD AND UNIT LOAD DATA

AMERICAN KITCHEN FOODS

Date	Unit	Total Amount	Unit Load/Ton Potato
4-20 to 4-21	BOD	30,800 lbs/d	70.0 lbs/t
1971	COD	42,500 lbs/d	96.6 lbs/t
	SS	25,200 lbs/d	57.2 lbs/t
	Flow	2.0± MGD	4,550 gal/t
4-22 to 4-23	BOD	30,200 lbs/d	68.6 lbs/t
1971	COD	41,500 lbs/d	94.4 lbs/t
	SS	30,600 lbs/d	69.5 lbs/t
	Flow	2.0± MGD	4,550 gal/t

TABLE 14

TOTAL LOAD - INFLUENT TO CARIBOU PLANT

Date	Unit	Total Amount
4-20 to 4-21	BOD	21,300 lbs/d
1971	COD	39,600 lbs/d
	SS	18,600 lbs/d
	Flow	4.0 MGD
4-22 to 4-23	BOD	20,400 lbs/d
1971	COD	27,800 lbs/d
	SS	14,300 lbs/d
	Flow	3.6 MGD

Analysis of Tables 13 and 14 indicate that the flow and organic loadings generated at the American Kitchen Foods plant, during the testing period, was somewhat higher than the industry average, and higher than many other plants in the area. It is reported that management is taking steps to reduce the plant loadings. It is also noted that the total organic load entering the Caribou treatment plant was about 30% less than the load generated at the American Kitchen Foods plant. While some BOD reduction may be expected in transit, it cannot account for the 30% reduction noted. Later system inspections revealed that one of the diversion manholes on the Utilities District interceptor line was silted up, allowing significant flow to be by-passed directly to the river. This interceptor carried the American Kitchen Foods waste flows. It is felt that this unintentional overflow accounts for a good portion of the 30% variance between the two points of testing, and that the data presented for the American Kitchen Foods plant was representative of conditions at the time of testing. It must also be recognized that the loads measured at the Caribou treatment plant did not represent the total load entering the river at Caribou.

During the summer of 1971, the American Kitchen Foods plant processed frozen peas. The data for a limited sampling on this waste stream is presented in Table 15.

TABLE 15

PEA PROCESSING WASTES - AMERICAN KITCHEN FOODS

	AKF Screen House	Caribou Treatment Plant
BOD, mg/l	1000	825
Temp., °F	83	80
Flow, MGD	2.2	2.8

While these loads are somewhat less than those generated by potato processing, they do represent a significant load. It is our understanding that American Kitchen Foods plans to discontinue pea processing in coming seasons.

A & P PROCESSING PLANT - Samples were composited on an 8 hour basis for 48 hour periods, from April 7, to April 10, 1971. Samples were taken on the raw plant effluent after screening, and after air flotation, which represents the loads entering the river Table 16 indicates the data after screening, and Table 17 shows effluent data after air flotation.

The waste characteristics presented in Tables 16 and 17 have been converted to total waste loads, and unit waste loads per ton of potato before and after air flotation. This total load data is presented in Tables 18 and 19.

TABLE 16

WASTE CHARACTERISTIC SUMMARY

A & P CO - RAW WASTES AFTER SCREENING

Date	Flow MGD	Ave BOD MG/L	Ave COD MG/L	Ave Total Solids MG/L	Ave Fixed TS MG/L	Ave SS MG/L
4-7 to 4-8 1971	0.818	2481	3217	4253	1035	1608
4-9 to 4-10 1971	0.752	2333	4080	4317	869	1845

Date	Ave Fixed SS MG/L	Ave Sett Sol ML/L	Ave Alka MG/L($CaCO_3$)	Chlorides MG/L	pH	Temp °F
4-7 to 4-8 1971	281	52	812	71	11.5	59°F
4-9 to 4-10 1971	154	44	687	66	11.5	49°F

TABLE 17

WASTE CHARACTERISTIC SUMMARY

A & P CO - WASTES AFTER AIR FLOTATION

Date	Flow MGD	Ave BOD MG/L	Ave COD MG/L	Ave Total Solids MG/L	Ave Fixed TS MG/L	Ave SS MG/L
4-7 to 4-8 1971	0.818	1869	2746	3423	1053	702
4-9 to 4-10 1971	0.752	1763	3007	3687	923	1048

Date	Ave Fixed SS MG/L	Ave Sett Sol ML/L	Ave Alka MG/L($CaCO_3$)	Chlorides MG/L	pH	Temp
4-7 to 4-8 1971	178	28	780	71	10.8	59°F
4-9 to 4-10 1971	161	17	725	--	10.8	59°F

Date	Total P MG/L	Soluble P MG/L	Ortho P MG/L	Ammonia N MG/L	Organic N MG/L
4-7 to 4-8 1971	19.8	7.6	2.6	0.015	0.000
4-9 to 4-10 1971	25.2	9.8	1.6	0.018	0.020

TABLE 18

WASTE LOAD SUMMARY

A & P CO - RAW WASTES AFTER SCREENING

Date	Unit	Total Amount	Unit Load/Ton Potato
4-7 to 4-8 1971	BOD	16,900 lbs/d	78.5 lbs/t
	COD	25,220 lbs/d	117.2 lbs/t
	SS	10,930 lbs/d	50.7 lbs/t
	Flow	0.817 mgd	3800 gal/t
4-9 to 4-10 1971	BOD	13,800 lbs/d	82.6 lbs/t
	COD	24,250 lbs/d	145.0 lbs/t
	SS	10,950 lbs/d	65.5 lbs/t
	Flow	0.752 MGD	4500 gal/t

TABLE 19

WASTE LOAD SUMMARY

A & P CO - WASTES AFTER AIR FLOTATION

Date	Unit	Total Amount	Unit Load/Ton Potato
4-7 to 4-8 1971	BOD	12,700 lbs/d	49.0 lbs/t
	COD	18,680 lbs/d	86.8 lbs/t
	SS	4,780 lbs/d	22.2 lbs/t
	Flow	0.818 MGD	3800 gal/t
4-9 to 4-10 1971	BOD	11,000 lbs/d	66 lbs/t
	COD	18,850 lbs/d	133 lbs/t
	SS	6,850 lbs/d	39.2 lbs/t
	Flow	0.752 MGD	4500 gal/t

The BOD data generated during the current studies correlate reasonably well with the limited data provided through the courtesy of the James W. Sewall Co, consultants to the Fort Fairfield Utilies District. The flows recorded during the recent survey suggest a 25 to 30 percent reduction in wastewater volume since the Sewall work in 1967.

The wastewaters from the A & P plant are generally typical of plants processing potatoes into french fries and potato by-products, utilizing the caustic peel process. Table 20 presents a comparison of average A & P loads per ton of potatoes processed to standard literature values.

TABLE 20

COMPARISON OF A & P WASTE WITH LITERATURE
(Raw Wastes - After Screening)

Item	A & P	Lit Low	Lit Avg	Lit High
BOD	80 lbs/t±	22 lbs/t	51 lbs/t	90 lbs/t
SS	60 lbs/t±	25 lbs/t	61 lbs/t	114 lbs/t
Flow	4000 lbs/t±	2310 g/t	4210 g/t	7000 g/t

Analysis of Table 20 indicates that the waste volume generated at the A & P plant is about, or slightly below, the industry average. The suspended solids value also appears to be about the industry average. However, the unit BOD production is somewhat above published industry average. This value reflects spring run conditions when the stored potatoes tend to be of lower quality, and peeling is somewhat more difficult. The processing plant was primarily utilizing the caustic peel process during the testing period. It was also noted that the potatoes processed during the testing period were smaller than normal. It is anticipated that fall conditions would indicate a lower unit BOD production. Other test data is typical of the industry.

During late July and early August the A & P plant processes frozen peas. While the loads generated during this operation are somewhat less than potato processing, they do represent a significant waste load. Test data from a limited 4 hour composite, during pea processing, is shown in Table 21.

TABLE 21

PEA PROCESSING - A & P PLANT

WASTEWATER ANALYSIS

August 4, 1971	Before Air Floatation	After Air Floatation
BOD, MG/L	795	360
Temp, °F	74	74
Flow, MGD	0.375	0.375

SEDIMENTATION EVALUATION

The wastewater testing program provides data for evaluation of primary treatment facilities applied to potato processing plants. The primary treatment facilities located at the processing plants on the Aroostook River are as follows:

Plant	Type Facility	Approximate Loading
Taterstate - Washburn	Rect Sedimentation	1935 gal/sf/d
Potato Service - P I	Cir Sed - Flume Water	520 gal/sf/d
Potato Service - P I	Cir Sed - Process Water	320 gal/sf/d
Amer Kit - Caribou	Rect Sedimentation	1100± gal/sf/d
Utilities Dist Plant		
A & P - Fort Fairfield	Air Flotation	1080 gal/sf/d

Data was obtained to evaluate the efficiency of these units on the basis of BOD, COD, and suspended solids removals. This evaluation is summarized in the following tables:

TABLE 22

TATERSTATE PROCESSING PLANT

Efficiency - Primary Treatment Unit

	4-6 to 4-7, 1971			4-7-4-9, 1971		
ITEM	IN-LBS	OUT-LBS	% EFF	IN-LBS	OUT-LBS	% EFF
BOD	24,400	19,000	22.1	29,600	24,700	16.6
COD	37,300	33,200	11.0	42,600	37,000	13.1
SS	20,800	16,050	22.8	24,700	17,050	31.0
		4-28-71				
BOD	21,050±	19,200±	8.8			

TABLE 23

POTATO SERVICE, INC

Efficiency - Flume Water Clarifier

	4-27 to 4-28, 1971			4-28 to 4-29, 1971		
ITEM	IN-MG/L	OUT-MG/L	% EFF	IN-MG/L	OUT-MG/L	% EFF
BOD	522	254	51.4	340	182	46.5
COD	1263	577	54.3	942	433	54.5
SS	2236	533	75.5	1519	286	81.4

TABLE 24

POTATO SERVICE, INC

Efficiency - Process Water Clarifier

	4-27 to 4-28, 1971			4-29, 1971		
ITEM	IN-MG/L	OUT-MG/L	% EFF	IN-MG/L	OUT-MG/L	% EFF
BOD	2893	2188	24.1	2793	2123	24.0
COD	5460	3745	31.4	5592	3453	38.2
SS	2828	391	86.2	2418	327	90.7

TABLE 25

CARIBOU TREATMENT PLANT

Efficiency - Primary Treatment Units

	4-20 to 4-21, 1971			4-22 to 4-23, 1971		
ITEM	IN-MG/L	OUT-MG/L	% EFF	IN-MG/L	OUT-MG/L	% EFF
BOD	660	390	40.7	590	435	26.3
COD	1100	610	35.4	865	640	26.0
SS	488	184	62.3	349	164	53.0

TABLE 26

A & P PROCESSING PLANT

Efficiency - Air Flotation Unit

Jordan Co Data - 1971

	4-7 to 4-8, 1971			4-9 to 4-10, 1971		
Item	IN-LBS	OUT-LBS	% EFF	IN-LBS	OUT-LBS	% EFF
BOD	16,900	12,700	24.8	13,800	11,000	20.3
COD	25,220	18,850	25.2	24,250	18,850	21.7
SS	10,930	4,780	56.3	10,950	6,550	40.3

Check with 1967 - Sewall Co Data

		3-16-67			3-23-67	
ITEM	IN-LBS	OUT-LBS	% EFF	IN-LBS	OUT-LBS	% EFF
BOD	12,600	9,700	23	15,500	10,800	29.4

TABLE 27

PRIMARY TREATMENT

Summary - All Units - Ave Efficiency

UNIT	BOD REMOVAL EFF	SS REMOVAL EFF
Taterstate - Sedimentation	16±%	26.9%
P S I - Flume Sed	49%	78.5%
P S I - Process Sec	24.1%	88.5%
Caribou T P	33.5%	57.5%
A & P - Air Flotation	24.5%	48.3%

The following comments can be made upon review of the foregoing data on various primary treatment units:

TATERSTATE - The existing primary unit at Taterstate is very ineffective. It is understood that the unit was intended to operate basically as an air flotation unit, with a recirculation pump and air inlet device. This portion of the facility was not operating effectively, and the unit was acting essentially as a plain sedimentation device. In addition to being loaded at over three times desired hydraulic surface loading, its configuration and method of solids removal leads to very low efficiencies. The unit is long and narrow (L=5W±), and has a water depth of only 4.5 feet. The settled solids and the floating solids are collected at the influent end of the tank, where they are partially removed by screw conveyors. No solids sump is provided. The solids that are removed from the unit are combined with screened solids for haul. In general, BOD removals above 10 to 15% cannot be expected from this unit.

POTATO SERVICE, INC - Both sedimentation units at Potato Service, Inc are of good design and appear reasonably well operated. The flume water clarifier is providing excellent BOD and solids removal. The process water clarifier is providing excellent solids removals, but on the test days, the BOD removal efficiency was somewhat below that anticipated. The reason for this relatively low BOD removal is not fully known, as the hydraulic loading of the unit is relatively low. The residence time of solids is quite high, as solids removal is retarded to achieve pH adjustment by biologic action to facilitate dewatering. The high residence time of solids during the test days could decrease BOD removal by leaching of soluable BOD from the solids. Previous data by Potato Service Inc personnel has indicated frequent BOD removals of from 40 to 50%. With careful control and operation, the clarifiers at Potato Service, Inc should generally obtain over 40% BOD removal.

CARIBOU TREATMENT PLANT - Hydraulic loadings to the primary clarifiers are quite high, ranging from 1.5 to 2 times the desired level. Even at this loading, however, moderate efficiencies are obtained when the units are in operation. The main treatment plant problem at this time is solids handling, which limits the time the plant can effectively operate. At the time of testing, about 8 hours of flow during each day is passed through the primary units, with the remainder being by-passed.

A plot of loadings to observed removals is shown on Figure 10. The plot of suspended solids removals to loading is quite consistent, while the BOD plot is more erratic, when the low observed efficiency at the Potato Service, Inc process clarifier is included. Based on previous detailed sedimentation studies at Taterstate[3], and literature studies, it is expected that with a surface loading of 600 gallons per square foot per day, and with effective settled solids removal, suspended solids removals of 80% or better, and BOD removals of 40% or better should be consistently obtained from a primary sedimentation tank. Figure 10 would confirm this effectiveness of primary sedimentation.

A & P - FORT FAIRFIELD - The existing air flotation unit is marginally effective as a primary treatment unit. Both BOD and suspended solids removals are below that desired for effective primary treatment. The unit appears to be operated reasonably well, and it is unlikely that

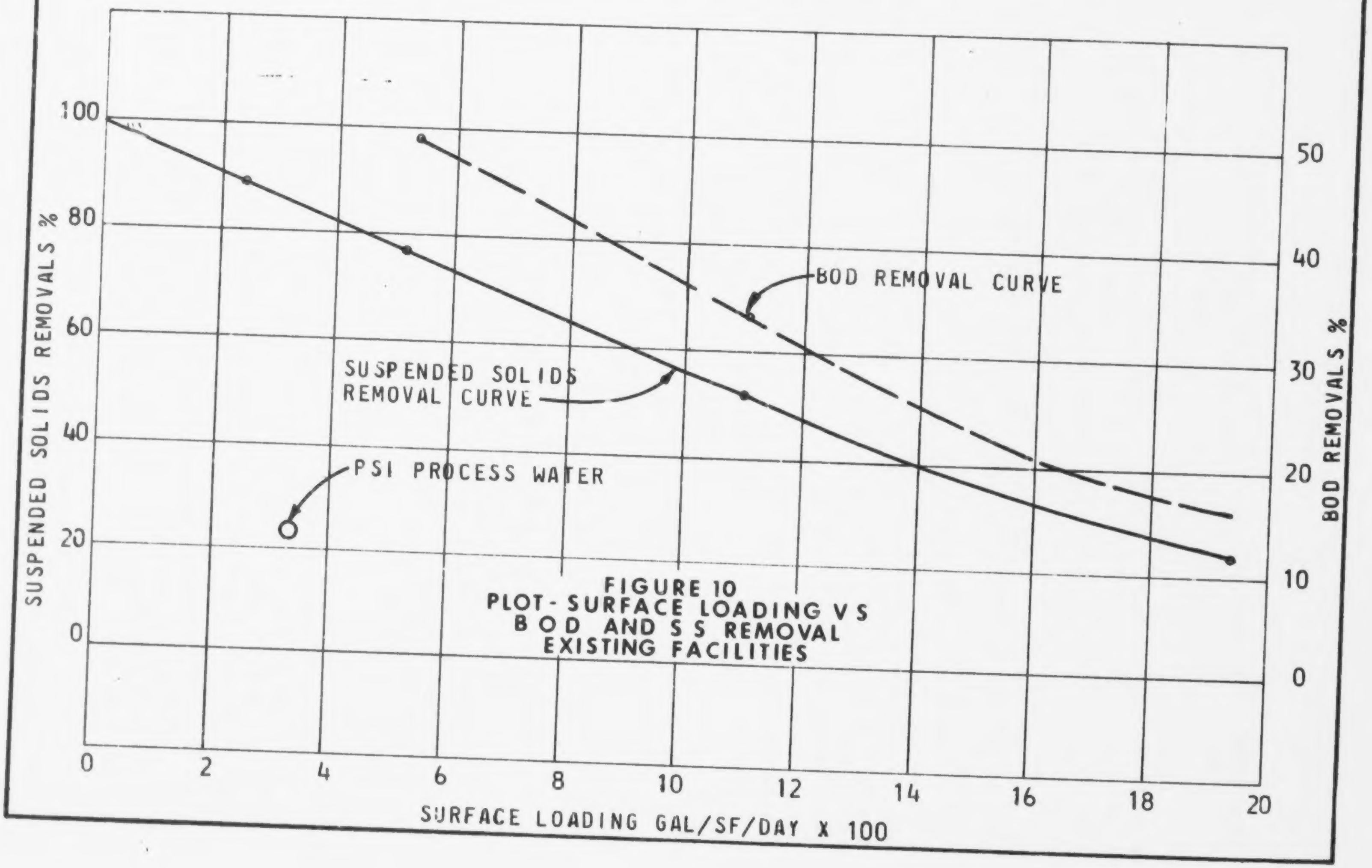

FIGURE 10
PLOT - SURFACE LOADING VS BOD AND SS REMOVAL EXISTING FACILITIES

consistantly higher removals can be obtained from this unit. It was also noted that the solids removed from the unit were quite wet, making handling and disposal difficult.

PRIMARY TREATMENT - EVALUATION OF NEED - Based on literature review and on evaluation of test data obtained during this study, it is concluded that primary treatment for the removal of potato solids, as close to the source as practical, is warranted. With good design and operation a primary treatment should generally remove over 40% of the BOD and over 80% of the suspended solids. Organic load removal of this magnitude, by primary treatment means, will be less costly than allowing it to pass to the biologic system.

Potato solids removal should be undertaken as close to the source as practical. The less time such solids are in the waste stream, the less organics will leach into solution. This time element will affect the BOD removal efficiencies of a primary treatment unit. It is also noted that potato solids removed by primary sedimentation can be used as an animal feed supplement. The potential value of this material may also enhance the value of early solids removal.

SECTION V

TREATMENT PLANT DESIGN CRITERIA

DESIGN WASTE LOADS

The foregoing sections discussed the general operating procedures at area processing plants and the waste loads generated in the spring of 1971. Evaluation of the research and development studies and preliminary design of pollution abatement facilities require the establishment of design loadings. The design loadings must include both the industrial loads, generated by local industry, and the domestic loads, generated by the resident population.

The industrial design loads, for the constructed facilities, may not necessarily be the same as the loadings established by the sampling and analyses presented in this report. Final design loadings must consider in-plant water and waste conservation measures, which may be undertaken by management and their effectiveness. The waste load reductions obtained through in-plant modifications must be balanced against possible production expansion over the life of any proposed facility. In the final analysis, each industry must designate the design waste loads which it proposes to contribute to a regional system. Once established, these values will represent that industry's share of available capacity, which cannot be exceeded without plant expansion, or agreements to reallocate capacity.

It is not possible, during the course of this study, to establish final design waste loadings for each industry. In order to evaluate systems, and to allow management to assess the cost impact of their waste load options, a range of potential loadings have been established. This range of loadings should reasonably represent the values which may be expected, if various in-plant adjustment options are exercised. It must be emphasized that these values are not rigid, or fixed at this time, but must be analyzed and refined as management evaluates its process. However, prior to final facility design, firm figures must be established for all industries.

The reader is referred to the companion Basin Planning Report for a more detailed discussion of the economic growth studies, which serve as a background for establishing the ranges of industrial wastewater flows.

The domestic wastewater loadings were based on 20 year population projections, developed in the companion Basin Planning Report, and upon data assembled from existing reports and operating data of municipal facilities. The selected design period of 20 years, for treatment facilities, was chosen as a reflection of a reasonable life expectancy of equipment associated with such facilities. Where new systems are proposed, a value of 100 gallons per capita per day and 0.20 pounds BOD per capita per day were used. It should be noted that modest variation in domestic flow will have little effect on preliminary system design.

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DESIGN LOAD CODING - Design loads, for the majority of communities included in this study, consist of both domestic and industrial waste loads. As outlined in Section IV, in-plant process modifications could alter the residual waste loads from each processing plant. Evaluation of external treatment of the residual wastewaters require estimates of the quantity and characteristics of the residual waste loads from the processing plants. A coding summary of loading conditions for the various processing plants, included in this report, is as follows:

Washburn Area Coding Summary:
  "A" Conditions - Taterstate loading alternatives
    A-1; Operation during Spring of 1971; no in-plant changes (maximum load)
    A-2; In-plant flow reductions and screen change (medium load)
    A-3; In-plant flow reductions, plus dry-caustic peel (low load)

  "B" Conditions - Taterstate product expansion alternatives
    B-1; Retain existing production capacity
    B-2; Expand with new flake line

Mapleton Area Coding Summary:
  "A" Conditions - Municipal wastes alone, no industrial wastes
    A-1; Municipal wastes alone

Presque Isle Area Coding Summary:
  "A" Condition - Loading alternatives at Potato Service Inc
    A-1; Operation during Spring of 1971, no in-plant changes (maximum load)
    A-2; In-plant flow reductions and solids recovery (medium load)
    A-3; Flow reductions, solids recovery, and dry-caustic peel (low load)

Caribou Area Coding Summary:
  "A" Conditions - Loading alternatives for combined American Kitchen Foods, Colby Starch, Cyr Brothers, and municipal wastewaters
    A-1; High industrial load options plus municipal wastewaters
    A-2; Medium industrial load options plus municipal wastewaters
    A-3; Low industrial load options plus municipal wastewaters

Fort Fairfield Area Coding Summary:
  "A" Conditions - loading alternatives at A & P
    A-1; Operation during spring of 1971, no in-plant changes (maximum load)
    A-2; In-plant flow reductions (medium load)
    A-3; In-plant flow reductions, plus dry-caustic peel (low load)

Easton Area Coding Summary:
"A" Conditions -Loading alternatives at Vahlsing, Inc
A-1; Estimated operation during Spring of 1971, no in-plant changes (maximum load)
A-2; In-plant flow reductions (medium load)
A-3; In-plant flow reductions, plus dry-caustic peel (low load)

"B" Conditions - Loading alternatives at Maine Sugar Industries
B-1; High industrial waste load condition
B-2; Medium industrial waste load condition
B-3; Low industrial waste load condition

The following paragraphs list in detail the range of loadings established for preliminary design analysis.

WASHBURN AREA - Wastewater loads in the Washburn area consist of the domestic wastewater of Washburn Village and Crouseville Village, and the process wastewater from Taterstate Frozen Foods. The Town of Washburn has accomplished considerable planning for municipal sewer system improvements and pollution control facilities to serve the Town. This work was accomplished by the Town's Consultants, the Edward C. Jordan Co, Inc in 1966.[3] The data generated in the above study has been reviewed in detail, updated, and serves as the basis for domestic design waste load projections. The domestic waste load projections made by the Town, in 1966, appear sound and should be considered valid input data by the regional study effort.

The domestic waste load generation for the Town of Washburn, as adopted for preliminary design and analysis, is shown in Table 28. It is noted that the projections include both the main urban area of Washburn and Crouseville Village, a part of the Town of Washburn to the east of the central village.

TABLE 28

TOWN OF WASHBURN - DOMESTIC WASTE LOAD

Design Year 1995

	Washburn Village	Crouseville Village
Population Served	2000	200
Design Flow (Average)	0.34 mgd	0.02 mgd
Biochemical Oxygen Demand	370 lbs/day	40 lbs/day
Suspended Solids	400 lbs/day	40 lbs/day

The only major generator of industrial wastewaters, currently operating in the Washburn area, is the Taterstate Frozen Foods processing plant.

This plant processes potatoes during the fall, winter, and spring months into frozen french fries and other frozen edible potato products. The plant is closed for a period of 60 to 75 days in the summer months.

In 1966, the Edward C. Jordan Co, Inc, assembled data on waste loads at the Taterstate plant, in the preparation of a report to the Town of Washburn.[3] This study indicated a wastewater flow of about 1.4 mgd, with a BOD of 8,000 to 9,000 pounds per day.. This data was utilized to develop preliminary design and cost estimates for primary treatment facilities. These facilities were expected to remove 40% of the BOD, and would produce an effluent capable of meeting the existing water classifications.

Subsequent to the 1966 Report, Taterstate has expanded its production facilities, and has installed limited pollution control units. The data presented in the previous section, indicates the load conditions as they existed in the spring of 1971.

The evaluation of possible internal system adjustments, as outlined in Section IV, at the Taterstate processing plant is primarily the responsibility of company management. However, to assist in their evaluation, and to allow preparation of alternate preliminary designs, an evaluation of the impact that the suggested changes herein would have on treatment of the residual wastes is presented The suggestedd in-plant changes must consider several factors, in addition to the impact on residual waste treatment. These are: capital cost of the change, increases or decreases in operating cost associated with the change, effect on product quality, effect on material recovery, and potential use of by-products.

Items 1, 2 and 3, as presented in Section IV, are primarily water conservation measures. Minimization of flow at various points within the plant could probably reduce wastewater flow to about 0.85 mgd (590 gpm), from current usage of about 1.0 mgd (695 gpm). However, continued surveillance and personnel education will be required to maintain the lower flow rate. This flow reduction, however, will not in itself significantly reduce total BOD, or suspended solids generation, as increased water recycle will tend to increase unit BOD and solids content.

Item 4 of the suggested in-plant adjustments; reduction of screen size, is primarily a minor equipment change and will require little expenditure of funds. The maintenance on this screen, and the collection and hauling of extra solids from the plant, will require additional in-plant operating effort. This procedure will reduce overall BOD and solids output from the plant, and will thus, reduce overall treatment costs.

Item 5; installation of a semi-dry caustic peel system to repl ace the existing facilities, will have the most impact on overall pollution reduction, both from the standpoint of flow volume and organic loading. It will also be the most difficult and costly to accomplish, if management should determine this procedure to be valid for their plant.

The semi-dry caustic process has been developed in recent years in an attempt to reduce the pollutional load from potato processing plants. This process consists of dipping the raw potatoes, after washing, in a heated caustic solution. After a penetration period, the potato is subjected to infra-red heat, after which the pulped skins are removed, as a paste material, by rubber brushes. The potato then receives a final wash, with relatively small amounts of water. In most instances, this wash water can be combined with the peel paste to form a slurry, which can be readily pumped for disposal.

The semi-dry caustic peel process greatly reduces, and may eliminate most wastewater flows from the peeling operation. Limited studies indicate the procedure produces a satisfactory product for further processing. Caustic usage with this process is no more, and probably less, than with the standard caustic peel process. The product recovery from the semi-dry process is reputed to be equal to other processes. Data on this factor may vary from plant to plant, and has not firmly established under actual plant operating conditions. If the infra-red process is used, the cost of fuel must be evaluated.

The peel pulp slurry generated in the system must be disposed of, or preferably utilized. As would be expected, this material will have a very high pH. After neutralization by bacterial fermentation, and mixed with other potato waste products, it has been used as cattle feed. Facilities for this fermentation process must be provided if the material is to be used as feed. If not fermented and used as feed, the disposal of this material could pose a problem. It cannot be simply dumped, without creating potentially serious nuisance conditions.

It is likely that new potato processing plants will adopt some variation of the semi-dry caustic peel operation. However, the conversion of older plants will require careful evaluation by management. Unfortunately, the current demands of pollution control are going to require decisions of management on adopting this system in the very near future; probably before full data on the process is available for evaluation.

The installation of cyclone-type solids concentrators on certain in-plant waste systems, as suggested in Section IV, may provide reasonable solids and BOD reductions. However, at this time, full evaluation of their effectiveness is not available.

Evaluation of external treatment of the residual Washburn wastewaters requires estimates of the quantity and characteristics of the residual waste load from the Taterstate processing plant. Table 29 presents the estimated residual waste loads under three in-plant conditions, based on current production capacity. These conditions reflect (1) no significant adjustment, (2) initiation of general water saving measures, together with screen mesh adjustment, and (3) conservation, as indicated in (2) plus conversion to the semi-dry caustic peel procedure.

TABLE 29

ESTIMATED RESIDUAL WASTE LOAD FROM TATERSTATE

BASED ON EXISTING PRODUCTION

Condition	Flow	BOD*	Suspended Solids*
A-1; B-1	1.00 mgd	300 mg/1-25,000 lbs/d	3000 mg/1-25,000 lbs/d
A-2; B-1	0.85 mgd	3000 mg/1-21,200 lbs/d	3000 mg/1-21,200 lbs/d
A-3; B-1	0.60 mgd	2100 mg/1-10,300 lbs/d	1600 mg/1- 8,000 lbs/d

*After Screening

The flows and characteristics indicated as Condition A-1; B-1 are essentially those measured during the recent sampling and gaging program. These are representative of spring conditions, which must be considered in any treatment plant design. The flows and characteristics of Condition A-2; B-1 reflect flow reduction, combined with limited reduction in total BOD and suspended solids by increasing screening efficiency, and possible installation of cyclone-type solid concentrators. Thus, the unit values of BOD and solids remain the same as Condition A-1; B-1, although flow and total BOD load is reduced.

Condition A-3; B-1, is an estimate of the impact of conversion to the semi-dry caustic peel process. Unfortunately, these values must be considered approximate at best, as full evaluation of the effects of this procedure on overall plant production is not complete, and hard data is not readily available. The estimates are based on limited studies that are currently available.

A study of the effect of infra-red peeling procedures, on secondary treatment, was prepared for the manufacturers of semi-dry caustic peel equipment.[30] This report presented an evaluation of waste loads produced by various peel procedures, including the semi-dry caustic peel. This study was based on literature surveys and data, with no actual plant wastewaters from the infra-red peel procedures. BOD entering the clarifier, using infra-red peeling, was estimated to be from 33 to 64 percent of that generated by conventional peel methods. The variation depends on the peel loss obtained in the standard peeling procedures. These percentages do not include the effect of initial potato fluming or wash wastewater. The inclusion of these wastes would lessen the percent reduction in wastes from the overall plant.

A study of the effect of the semi-dry caustic peel procedure on waste generation is currently underway by Western Potato Service in Idaho, supported by the Federal Water Quality Administration of the Environmental Protection Agency (Project FWPCA 12060 EIG). A progress report summary was issued in March, 1971. The data presented in the progress report is limited. However, it does suggest total plant loads of about 40 lbs BOD and about 1670 gallons of process water per ton of potatoes processed, with the dry caustic peel process. As this data is based on early operation of equipment at the plant, a further reduction in BOD may be anticipated. The design values established for the Washburn analysis are about 32 lbs BOD and 1800 gallons per ton of potatoes processed. This represents a BOD reduction of 60% from the current BOD generation. It is felt that these values are representative of the effect of conversion to the semi-dry caustic peel process, based on available data.

Taterstate management has indicated it is considering expansion of facilities to provide a potato flake line at Washburn. This line could have a potato input capacity of up to 90 tons per day. If this expansion should occur, the loading conditions at Taterstate would increase to the values shown in Table 30, for the three assumed in-plant conditions, as previously outlined.

TABLE 30

TATERSTATE LOADINGS - WITH FLAKE LINE ADDED

Condition	Flow	BOD	Suspended Solids
A-1; B-2	1.27 mgd	3000 mg/1-31,700 lbs/d	3000 mg/1-31,700 lbs/d
A-2; B-2	1.10 mgd	3000 mg/1-27,500 lbs/d	3000 mg/1-27,500 lbs/d
A-3; B-2	0.77 mgd	2100 mg/1-13,400 lbs/d	1600 mg/1-10,300 lbs/d

MAPLETON AREA - Mapleton Village has a small sanitary sewer system, serving only a portion of the Town. The Mapleton Sewer District has proposed expansion of this system to serve the entire Village area. If this is to be accomplished, treatment facilities must be provided. To allow analysis of a regional system, domestic waste flow estimates have been prepared and are presented in Table 31. There are no significant industrial waste loads in the Mapleton area.

TABLE 31

TOWN OF MAPLETON - DOMESTIC WASTE LOAD

Design Year 1995

Population Served	800
Design Flow (Average)	.08 mgd
Biochemical Oxygen Demand	160 lbs/d
Suspended Solids	160 lbs/d

PRESQUE ISLE AREA - Wastewater loads in the Presque Isle area consist of the domestic wastewater from the City of Presque Isle, and the process waters from the Potato Services, Inc, potato processing plant. The Presque Isle Sewer District has maintained a primary treatment facility for the City's domestic wastewater for a number of years. Data and existing reports from the District allow reasonable projection of the domestic waste loads. These projections are shown in Table 32.

TABLE 32

CITY OF PRESQUE ISLE - DOMESTIC WASTE LOAD

Design Year 1995

Population Served	14,000
Design Flow (Average)	1.40 mgd
Biochemical Oxygen Demand	3,000 lbs/d
Suspended Solids	3,000 lbs/d

The major industrial wastewater generator in the Presque Isle area is the Potato Service, Inc, processing plant. This plant operates during the fall, winter and spring, with a down period of approximately 60 days in the summer. Data presented in Section IV indicates the waste load conditions present in the spring of 1971.

For preliminary design and analysis, three loading options have been established. Condition A-1 reflects essentially the measured conditions. Condition A-2 anticipates recycling about 100,000 gpd clarified flume water, general tightening up of in-plant water use and controls, and installation of cyclone-type solids recovery units on some waste streams. These procedures were presented and discussed in Section IV. Condition A-3 assumes the general plant improvements under Condition A-2, plus full conversion to the dry-caustic peel process. The discussion on dry caustic peel procedures, as presented for Taterstate in Washburn, are equally applicable at all plants.

Both raw waste loads (after screening) and waste loads leaving the existing clarifiers have been estimated. Estimates have been made for the flume line and process line separately and the combined effluent. This data is presented in Table 33.

CARIBOU AREA - The wastewater generated in the Caribou area currently consists of the domestic wastewaters of the City, and the process wastewaters of American Kitchen Foods and Colby Cooperative Starch Co. The potato processing wastes of Cyr Bros is expected to go on line shortly. In addition to these firm units, the Caribou Development Corporation is attempting to establish a new processing plant in the Grimes Mill area of Caribou. As the success of this expansion effort may be tied directly to solution of the area's pollution control problem, load projections

TABLE 33

WASTE LOAD PROJECTIONS - POTATO SERVICE, INC

Condition		RAW LOAD			LOAD - PRIMARY EFF		
		Flume	Process	Total	Flume	Process	Total
A-1							
	Flow	0.5 mgd	2.6 mgd	3.1 mgd	0.5 mgd	2.6 mgd	3.1 mgd
	BOD	2,290 lbs/d	62,800 lbs/d	65,100 lbs/d	1,170 lbs/d	47,700 lbs/d	48,870 lbs/d
	SS	9,000 lbs/d	61,200 lbs/d	70,200 lbs/d	2,000 lbs/d	8,500 lbs/d	10,500 lbs/d
A-2							
	Flow	0.4 mgd	2.5 mgd	2.9 mgd	0.4 mgd	2.5 mgd	2.9 mgd
	BOD	1,900 lbs/d	60,400 lbs/d	62,300 lbs/d	950 lbs/d	41,650 lbs/d	42,600 lbs/d
	SS	7,500 lbs/d	57,000 lbs/d	65,500 lbs/d	1,500 lbs/d	7,000 lbs/d	8,500 lbs/d
A-3							
	Flow	0.4 mgd	1.6 mgd	2.0 mgd	0.4 mgd	1.6 mgd	2.0 mgd
	BOD	1,900 lbs/d	27,800 lbs/d	29,400 lbs/d	950 lbs/d	17,400 lbs/d	18,350 lbs/d
	SS	7,500 lbs/d	20,000 lbs/d	27,500 lbs/d	1,500 lbs/d	3,500 lbs/d	5,000 lbs/d

have been prepared with, and without this new plant. It should be noted that the policy of the Caribou Utilities District provides for joint treatment of all industrial and domestic wastewaters in its system. However, for clarity the flow projections have been subdivided.

The design domestic waste loads for the City of Caribou are presented in Table 34.

TABLE 34

CITY OF CARIBOU - DOMESTIC WASTE LOAD

DESIGN YEAR 1995

Population Served	12,000
Design Flow (Average)	1.2 mgd
Biochemical Oxygen Demand	2,400 lbs/d
Suspended Solids	2,400 lbs/d

Three industrial waste load conditions have been established for the combined in-town plants, ie, American Kitchen Foods, Colby Starch, and Cyr Bros. These loadings are shown in Table 35.

Condition A-1 assumes the American Kitchen Foods and Colby loads will remain at their maximum recorded in the spring of 1971. The Cyr Bros load assumes production of about 1500 lbs/hr of product, with only limited success in its in-plant water management program. Thus, Condition A-1 would represent the maximum expected waste load generation from in-town Caribou.

Condition A-2 assumes full conversion to the dry-caustic peel system at American Kitchen Foods. Colby Starch is assumed to achieve limited in-plant load reductions. Cyr Bros is assumed to have a production of 10,000 lbs/hr of product and moderate success in its in-plant water management program. This condition would reflect a reasonable median overall waste load.

Condition A-3 again assumes American Kitchen Foods converts to the dry-caustic peel system. Colby is assumed to achieve a substantial waste reduction through elimination of its protein water component. The Cyr Bros' plant production rate is assumed to be 10,000 lbs/hr of product, with substantial success in its water management program. This condition would appear to reasonably represent the minimum residual flow conditions that could be expected from the in-town Caribou plants.

It must be recognized that a nearly unlimited number of intermediate load conditions could be realized, by differing the success of each plant's water conservation program. While it is not possible to study each possible combination, the above conditions should represent a range within

TABLE 35

INDUSTRIAL LOAD ALTERNATIVES

IN-TOWN CARIBOU*

CONDITION	LOAD UNIT	AKF	CYR	COLBY	TOTAL
A-1	Flow	2.2 mgd	0.7 mgd	0.5 mgd	3.4 mgd
	BOD	31,000 lbs/d	12,000 lbs/d	20,000 lbs/d	63,000 lbs/d
	SS	30,000 lbs/d	4,000 lbs/d	10,000 lbs/d	44,000 lbs/d
A-2	Flow	1.0 mgd	0.4 mgd	0.5 mgd	1.8 mgd
	BOD	15,000 lbs/d	5,000 lbs/d	18,000 lbs/d	38,000 lbs/d
	SS	12,000 lbs/d	2,000 lbs/d	8,000 lbs/d	32,000 lbs/d
A-3	Flow	1.0 mgd	0.2 mgd	0.3 mgd	1.5 mgd
	BOD	15,000 lbs/d	3,000 lbs/d	5,000 lbs/d	23,000 lbs/d
	[illegible]	12,000 lbs/d	1,000 lbs/d	2,000 lbs/d	15,000 lbs/d

*Prior to primary sedimentation

which actual loadings will fall. Prior to final design of the treatment facilities, each processing plant must designate its desired load input, allowing determination of actual total load values.

In addition to the in-plant industrial loads, the analysis includes consideration of a significant load at Grimes Mill. This allowance has been set at 1.4 mgd and 17,000 lbs/d BOD. These loads would allow installation of a relatively large processing plant at Grimes Mill, assuming that any new plant would utilize the latest techniques in plant design and water conservation programs.

FORT FAIRFIELD AREA - The waste loads generated in the Fort Fairfield area consist of the domestic wastewater of the urban area, and the process wastewater from the A & P Company processing plant. The Fort Fairfield Utility District has accomplished considerable planning for a municipal sewer system and pollution control plant to serve the Town of Fort Fairfield. This work was accomplished by the District's Consultants, The James W. Sewall Co. The data generated in these studies has been reviewed in detail and serve as the basis for design waste load projections. The domestic waste load projections made by the District appear sound and should be considered valid input data to the regional study effort. The domestic waste load generation for the Town of Fort Fairfield as adopted for preliminary design and analysis is shown in Table 36.

TABLE 36

FORT FAIRFIELD - DOMESTIC WASTE LOAD

	Design Year 1995
Population Served	3,500
Design Flow (Average)	0.35 mgd
Biochemical Oxygen Demand	700 lbs/d
Suspended Solids	700 lbs/d

The only major generation of industrial wastewaters, currently operating in the Fort Fairfield area, is the A & P processing plant. This plant processes potatoes during the fall, winter, and spring months into frozen french fries and other frozen edible potato products. During a period in the summer, the plant processes peas for freezing. The more significant waste loads are generated during the potato processing function. These will be the critical design loads. The design loads for three alternative in-plant conservation conditions are shown in Table 37.

The flows and characteristics indicated as Condition A-1 are essentially those measured during the recent sampling and gaging program. These are representative of spring conditions, which must be considered in any treatment plant design.

TABLE 37

ESTIMATED RESIDUAL WASTE LOAD FROM A & P

In-plant Adjustment	Flow	BOD*	Suspended Solids*
Condition A-1	0.82 mgd	2500 mg/1-17,100 lbs/d	2000 mg/1-13,670 lbs/d
Condition A-2	0.60 mgd	3400 mg/1-17,000 lbs/d	2700 mg/1-14,600 lbs/d
Condition A-3	0.40 mgd	2100 mg/1- 7,000 lbs/d	1600 mg/1 5,300 lbs/d

*After Screening

The flows and characteristics of Condition A-2 reflect essentially a flow reduction with little or no reduction in total BOD or suspended solids.

Condition A-3 is an estimate of the impact of conversion to the semi-dry caustic peel procedure. The discussions and comments on this peeling process, under Washburn, are equally applicable to Conditions at Fort Fairfield.

EASTON AREA - The total waste load generated in the Easton area will consist of the domestic loads of the Town of Easton, plus the industrial wastes from the Vahlsing, Inc, potato processing plant and the Maine Sugar Industries sugar refining plant. At the present time, the Town of Easton does not have a sanitary sewer system. However, soil conditions are such that continued long term use of individual subsurface disposal facilities may not be possible and a central system will be required. Thus, estimates of the domestic load have been made for use in the preliminary design analysis. The estimated domestic loads are given in Table 38, and are based in part on the 1970 study prepared by Camp Dresser and McKee[33].

TABLE 38

EASTON - DOMESTIC WASTE LOAD

Design Year 1995

Population Served	1020
Industrial Employees	1500
Flow (Average)	0.30 mgd
Biochemical Oxygen Demand	460 lbs/d
Suspended Solids	460 lbs/d

It is pointed out that the domestic waste loads in Easton are vitally affected by the industrial growth of the Town. The above data assumes a favorable climate and the reestablishment of a viable sugar industry. However, if design of treatment facilities for only the domestic waste were to be undertaken, it should be designed for lower loads, with ease of expansion built in.

Three alternative loading conditions have been established for both the Vahlsing potato processing plant and the Maine Sugar Industries plant. The loadings at the Vahlsing plant are based on a potato input capacity of 285 tons per day. The loadings at the Maine Sugar Industries plant are based on a capacity of 4000 tons of beets per day. These capacities are as reported in the 1970 Camp Dresser and McKee report[33]. A summary of the design loadings is given in Table 39.

TABLE 39

INDUSTRIAL LOADINGS - EASTON AREA

Vahlsing Inc	Flow	BOD	SS
Condition A-1	2.1 mgd	25,000 lbs/d	25,000 lbs/d
Condition A-2	1.4 mgd	20,000 lbs/d	18,000 lbs/d
Condition A-3	0.8 mgd	13,000 lbs/d	10,000 lbs/d
Maine Sugar Industries			
Condition B-1	2.8 mgd	16,000 lbs/d	15,000 lbs/d
Condition B-2	2.0 mgd	10,000 lbs/d	10,000 lbs/d
Condition B-3	1.2 mgd	5,000 lbs/d	5,000 lbs/d

Condition A-1 in the above listing assumes normal processing conditions with loads as reported in the 1970 Camp Dresser and McKee report[33]. These values are higher than industry averages, and would likely be the maximum waste load production that would be anticipated. Condition A-2 would reflect a general tightening up of plant procedures, possible use of cyclone-type solids concentrators, but with standard lye peel retained. Condition A-3 would reflect tightened flow conditions, as with Condition 2, plus conversion to dry-caustic peel procedures.

Condition B-1 at Maine Sugar Industries reflects loadings suggested in the 1970 Camp Dresser and McKee report[33], and would likely represent the maximum waste loads to be realized from the modern facility at Easton. Condition B-2 assumes a closure of the internal systems and general good processing practice. Condition B-3 assumes nearly complete system closure, and probably reflects the lowest load which can be consistently maintained at the plant.

Again it must be emphasised that the above represent a range of values, and many other combinations are possible. Prior to final design each plant must evaluate its internal practices to determine its final design loading.

COMBINED SEWER CONSIDERATIONS - The existing sewer systems in all communities are combined in nature to some degree. This condition will affect the hydraulic designs of treatment facilities. The Town of Washburn proposes to separate all surface water from its sanitary system prior to construction of pollution control facilities. This should effectively minimize the impact of surcharge flows on the Washburn system.

The City of Presque Isle system is combined and will likely remain so for some time. In the long term it may be desirable to separate this system. However, the economic realities of the situation will delay this effort for an extended period of time. Thus, during periods of high runoff, discharges of combined flow will enter the Presque Isle Stream in the vicinity of the existing treatment plant.

The Caribou Utilities District System is also combined, and like Presque Isle, is likely to remain so for some time. Again long range plans may call for ulitmate separation, if funds were available. Until this is done overflows of combined wastes will enter the Aroostook River through various overflow structures in the District's system.

The proposed water pollution control program by the Fort Fairfield Utilities District provides for extensive new sewer construction. This work will be as a separate sewer system. With this program, it is felt that the Fort Fairfield system can be effectively separated. Likewise, the new systems proposed at Easton and Mapleton will be constructed to carry sanitary wastewaters only.

INDUSTRIAL EXPANSION - The range of loadings given herein for the industrial plants in the region are based essentially on current production capacity, except for the Taterstate plant where management has indicated a definite expansion program. Future expansion of the potato processing industry in Aroostook County is considered essential to the economy of the region, and economic studies undertaken in the companion River Basin Planning study support the probability of reasonable growth. Yet it is not possible at this time to predict the total expansion, its timing, and its location. It may occur through expansion of existing plants, such as proposed in Washburn, or at new locations such as suggested at Grimes Mill in Caribou.

With the vagaries of an agricultural industry, it does not appear reasonable to design large amounts of surplus capacity into a system unless it is reasonably certain expansion will occur. Fortunately, the changing technology of the potato processing industry may aleviate this problem to some degree. As new peeling processes are developed, it is anticipated that unit waste load generation per ton of potatoes will drop significantly. New plants will almost certainly be equipped with such equipment and a gradual conversion may be anticipated in older plants. Thus, it is quite likely that increased production capacity can be achieved with total waste loads not significantly exceeding 1971 levels. It is suggested that consideration should be given to constructing pollution control facilities to handle reasonably high loads, in the ranges presumed. If this is done, it is likely that increased production capacity in the future can be handled within the system. Where possible, facilities should be designed to accept future expansion as easily as possible.

TREATMENT DESIGN CRITERIA

In order to properly evaluate the research and development studies, in relation to their impact on the study area, preliminary treatment system designs, to handle the range of loads presented herein, have been prepared. A review of the water quality studies, conducted under the companion Basin Planning report, indicates that at least 85% BOD removals will be required for all significant waste loads. As current State and Federal policy calls for secondary treatment of all wastes, this report assumes at least 85% BOD removals for all wastes.

Nearly all of the biologic treatment studies on potato processing wastes have been of the activated sludge process, or some variation thereof. A review of the literature has been completed and is summarized in Section XI. Based on this review, it is apparent that activated sludge technology will be well suited to the treatment of potato processing wastes, or potato processing wastes in combination with domestic waste flows.

Based on literature studies, proposed treatment design criteria have been assembled. While sufficient for preliminary design analysis, some refinement will be necessary before final design. A summary of key preliminary design criteria is given in Table 40.

TABLE 40

SUMMARY - TENTATIVE DESIGN CRITERIA

POTATO PROCESSING WASTEWATER TREATMENT

1. PRIMARY SEDIMENTATION

A. Surface Loading = 600 gal/sf/day
B. Side Water Depth = 10 to 12 feet
C. BOD removal efficiency - Average 40% to 45%
D. SS removal efficiency - Average 85%
E. Solids removed at 5 to 6% - Steam Peel
at 3 1/2 to 5% - Caustic Peel
Design - 4% Solids
with thickening in clarifier - 7%
F. Rate of solids removal - Adjust to have pH of solids between 6.0 and 7.0

2. DEWATERING PRIMARY POTATO SOLIDS

A. Utilize vacuum filtration
B. Filter loading - 5 lbs/sf/hour

TABLE 40
(CONTINUED)

C. Operate 24 hrs/day to dewater at optimum pH
D. Design for installation of conditioning chemicals, if needed
E. Solids content of cake = 9 to 14% w/o conditioning
    = 17 to 19% with conditioning
F. Filtrate = 2500 mg/l solids w/o conditioning
    = 1000 mg/l solids with conditioning

3. COMPLETE MIXED ACTIVATED SLUDGE

A. Aeration Volume: The larger as computed by
    1. Detention Time = 12 hours based on inflow plus 30% solids return, or
    2. Organic Load = 180 lbs BOD per 1000 cf aeration volume
B. Design MLSS = 3000 mg/l to 4000 mg/l - resulting in BOD to MLSS ratio of less than 0.75 to 1
C. Reaction Rate K_s = 0.000425 at 20°C
D. Solids Production = (a) (BOD removed) - (b) (MLVSS)
    a = 0.65   b = 0.05   VSS = 0.8 SS
E. Secondary Clarifier - 600 gal/sf/day based on inflow plus 30% solids recycle
F. Solids Underflow = 1.5%
H. Oxygenation Capacity = 1 lb O_2 per lb BOD removed
I. Anticipated BOD Removal = 90%

4. EXCESS BIOLOGIC SOLIDS REMOVAL

A. Vacuum Filtration
B. Thicken to 4% to 6% gravity or air flotation thickening prior to filtration
C. Filter Loading - 4 lbs/hr/sf
D. Provide for chemical conditioning
E. Solids content in cake = 15%
F. Filtrate = 1500 mg/l Solids.

5. DISINFECTION

A. Chlorination - Design Normal Application = 10 mg/l
    - Capacity = 30 mg/l
B. Residual Desired = 2.0 mg/l
C. Contact Time = 15 minutes at peak hourly flow
D. Dual Units - each with full capacity

The above criteria apply directly to treating potato processing wastes alone. However, they are essentially applicable to a reasonable combination of processing and municipal wastes. The municipal wastewaters would not be subjected to primary treatment, but would be introduced directly into the aeration units. Otherwise, all of the above criteria were considered applicable. This is a conservative approach, but is valid for a comparative analysis.

Treatment of the domestic wastewaters alone would be accomplished by the standard extended aeration process in the smaller communities, and by standard activated sludge in the larger communities, with criteria as established in the "10 State Standards," so called, as adopted by the Maine Environmental Improvement Commission.[37]

SECTION VI

TREATMENT-TRANSPORT SYSTEM ANALYSIS

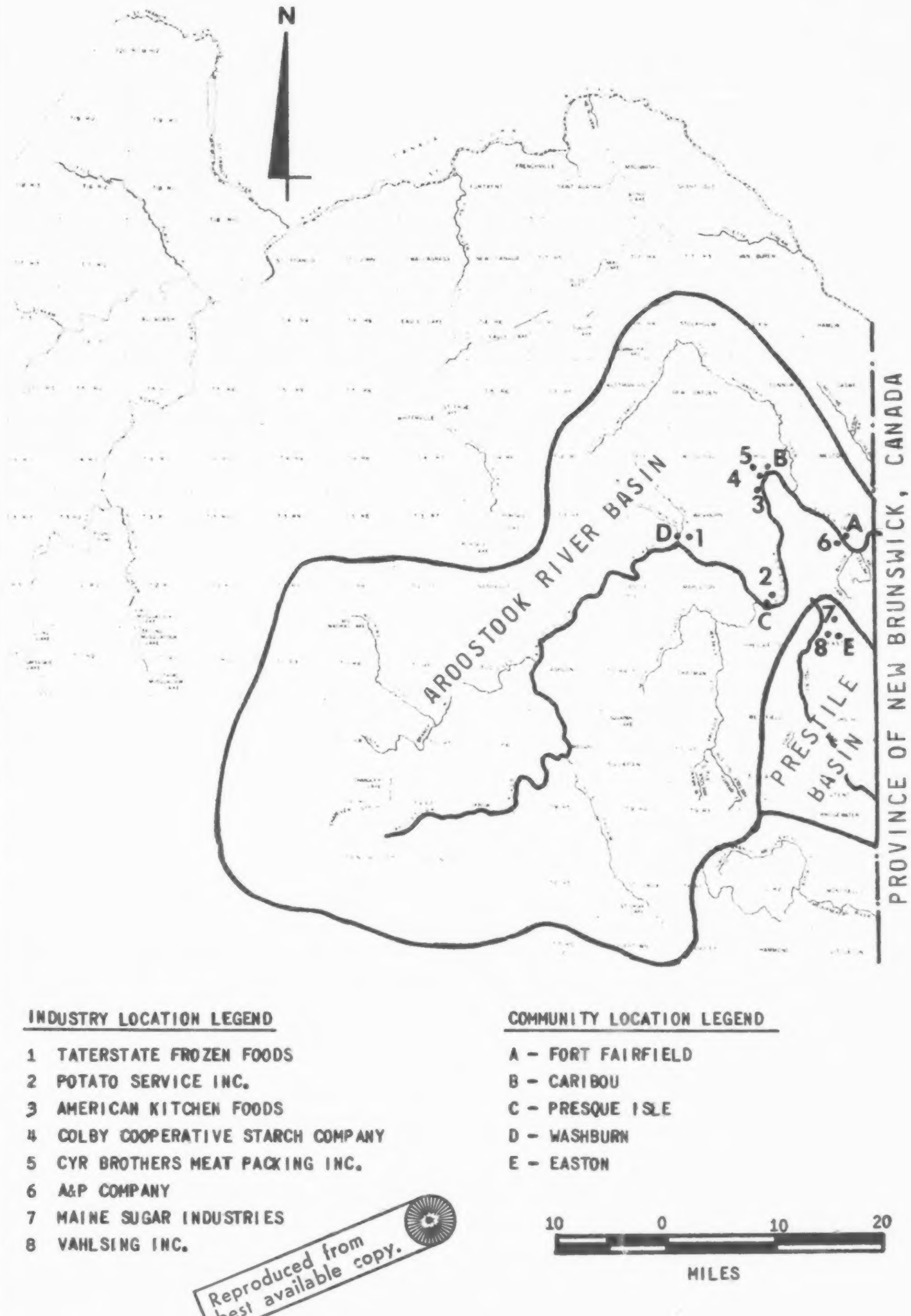
INTRODUCTION - The Aroostook River Basin has major population and industrial facilities located at Fort Fairfield, Caribou, Presque Isle, and Washburn, while the adjacent Prestile Stream Basin has major industrial facilities located in Easton. The location of each community and industry, in each Basin, is shown in Figure 11.

The economy of each Basin is strongly agriculturally oriented, with potatoes the dominant crop. On the Aroostook Basin, agriculture and potato processing are concentrated in its lower reaches with the headwater areas being wild lands in timber management. The Prestile Basin, however, has agriculture and processing facilities concentrated in its headwater area.

Since the early 1950's the potato economy of the nation, and Aroostook County, has evolved from predominantly a fresh pack market to a substantial processed product market. Currently about 40% of the Aroostook crop is processed, primarily into frozen french fires and byproducts. The trend toward processing a larger portion of the crop is expected to continue into the 1970's.

Processing of a substantial portion of the potato crop has created an increasingly serious pollution problem in the Aroostook River and adjacent Prestile Stream. The organic waste loads now generated by potato processing plants in the area represent a load nearly thirty times that generated by the resident population. This condition has created nuisance conditions in both the Aroostook and Prestile waters, and represents a frequent violation of State and Federal water quality standards.

In the mid 1960's it became evident that further expansion of the processing industries of northern Maine was vitally dependent upon solving the severe pollution problems of the area, consistent with the economic resources of the communities and industries of the Basins. To accomplish this objective, the Northern Maine Regional Planning Commission initiated a preliminary evaluation of the problem which culminated in a general comprehensive water resources plan for the Aroostook-Prestile Basins[34]. This initial report recommended that the Aroostook-Prestile Basins be considered as a hydrologic unit, and that all pollution control efforts be coordinated on a river basin planning basis. The report also suggested the physical integration of water pollution control facilities, serving the lower Aroostook Basin and upper Prestile Basin, could achieve the maximum degree of water quality improvement at minimum cost. These early studies indicated that the geographical and topographic configuration of the lower Aroostook Basin may permit utilization of a combined treatment-transport system to achieve regional system economies.


N
AROOSTOOK RIVER BASIN
PRESTILE BASIN
PROVINCE OF NEW BRUNSWICK, CANADA
1
2
3
4
5
6
7
8
A
B
C
D
E
10
0
10
20
MILES

INDUSTRY LOCATION LEGEND

1 TATERSTATE FROZEN FOODS
2 POTATO SERVICE INC.
3 AMERICAN KITCHEN FOODS
4 COLBY COOPERATIVE STARCH COMPANY
5 CYR BROTHERS MEAT PACKING INC.
6 A&P COMPANY
7 MAINE SUGAR INDUSTRIES
8 VAHLSING INC.

COMMUNITY LOCATION LEGEND

A - FORT FAIRFIELD
B - CARIBOU
C - PRESQUE ISLE
D - WASHBURN
E - EASTON

Reproduced from best available copy.

FIGURE 11
COMMUNITY AND INDUSTRY LOCATION PLAN

The treatment-transport system, as initially proposed, is shown in Figures 12A and 12B. The original plan propcsed transporting the waste activated sludge solids from the Washburn area, and the wastewater flows from the Easton area, to Presque Isle. At Presque Isle the waste flows, from both Washburn and Easton, would be combined with the wastewater flows from Presque Isle. These flows would then be conveyed to Caribou via the treatment-transport system. A terminal treatment facility was also proposed at Caribou for treatment of the wastewater flows generated in Caribou, and the flows discharged from the treatment-transport system.

The Northern Maine Regional Planning Commission adopted the recommendations of the initial report as the basis for subsequent pollution control planning in the Aroostook-Prestile Basins. The Commission retained the Edward C. Jordan Company, Consulting Engineers and Planners, to organize an extensive planning effort which would lead to a Basin-wide program of pollution abatement. The proposed preliminary design and planning effort was divided into two basic phases, research and development, and Basin planning; with one coordinated with the other to assure implementation of the program at the earliest possible date.

The research and development phase of the project was primarily directed at establishing the feasibility of constructing the treatment-transport system between Presque Isle and Caribou. In order to accomplish a relatively constant degree of treatment in such a system, with widely varying flows, uniform velocities of flow, and thus transit time, must be maintained. If this can be accomplished, the problems of oxygen transfer to the system, and maintaining an active biological growth must be evaluated.

Evaluation and design of a biological treatment-transport channel system requires analysis of the interrelation of channel hydraulics, aeration parameters, and biologic parameters.

The original program included a pilot plant study to establish the detailed design criteria for a treatment-transport system. This work was omitted, however, from the research and development program.

The following paragraphs present the studies and evaluations undertaken for development of the treatment-transport system; namely, system configuration, hydraulic, oxygen transfer, and biological. A general discussion on the proposed treatment-transport system is also included.

SYSTEM CONFIGURATION - Prior to undertaking detailed studies on the treatment-transport system, it was necessary to confirm the system's configuration. Initial studies were undertaken to determine the feasibility of transporting the waste solids from Washburn and the wastewater flows from Easton to the Presque Isle area, as previously discussed. These studies, presented in Section VII, conclude that the waste solids generated in the Washburn area should be treated in a separate facility located in Washburn, while the Easton wastewater flows could reasonably be transported to the Presque Isle area.

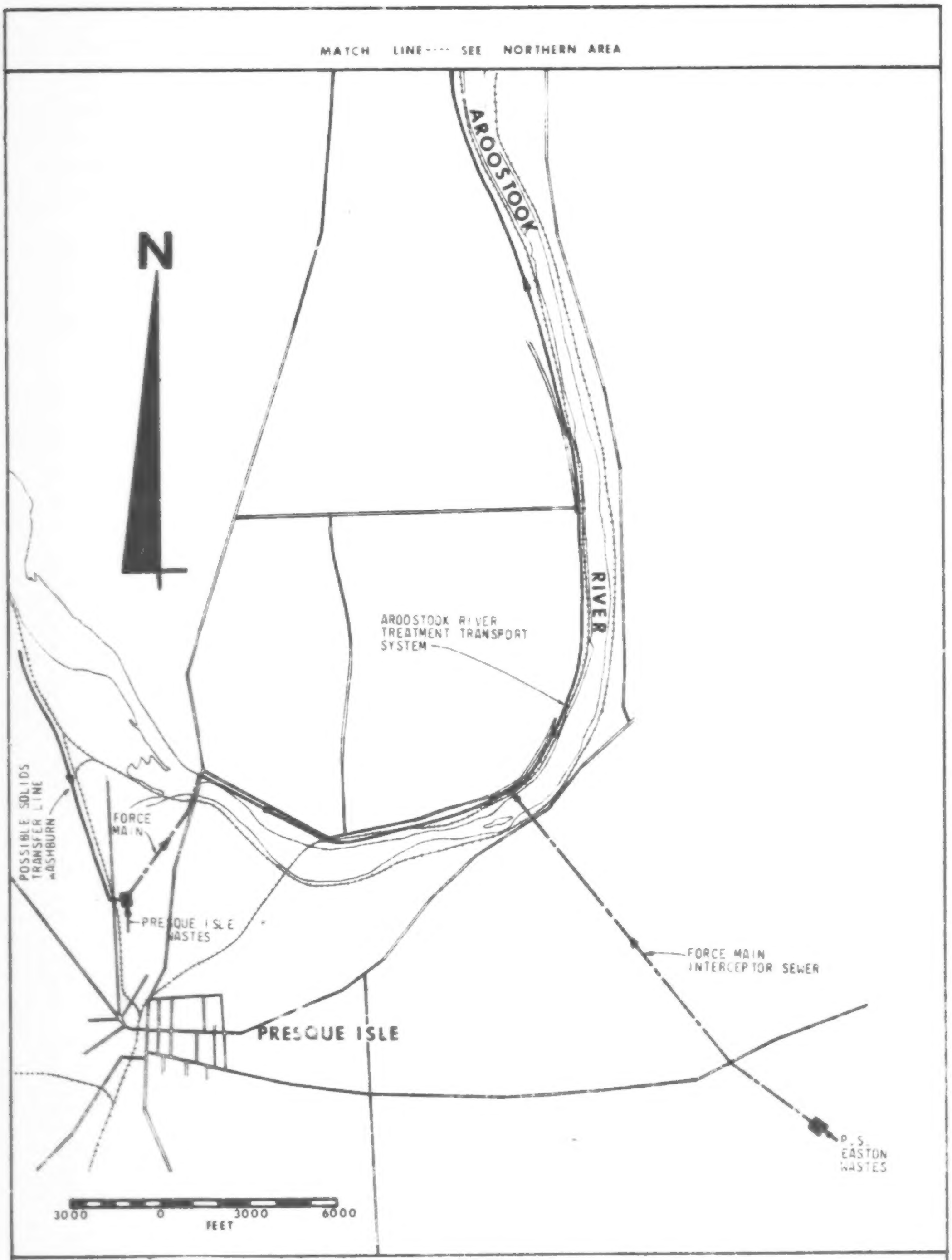

MATCH LINE---- SEE NORTHERN AREA
N
AROOSTOOK
RIVER
AROOSTOOK RIVER TREATMENT TRANSPORT SYSTEM
POSSIBLE SOLIDS TRANSFER LINE WASHBURN
FORCE MAIN
PRESQUE ISLE WASTES
FORCE MAIN INTERCEPTOR SEWER
PRESQUE ISLE
P.S. EASTON WASTES
3000 0 3000 6000
FEET

FIGURE 12 A
INITIALLY PROPOSED TREATMENT TRANSPORTATION CONFIGURATION
SOUTHERN AREA

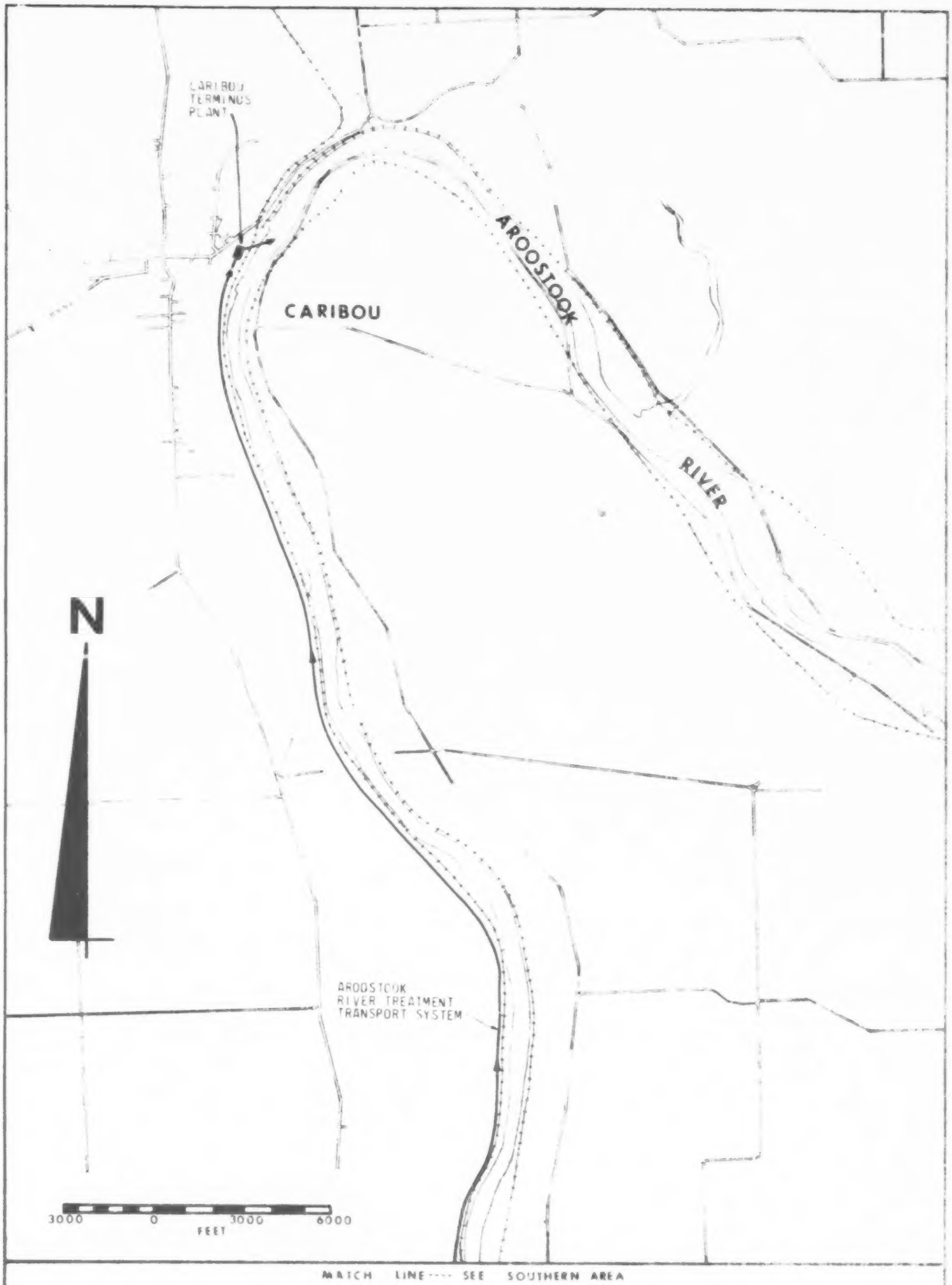

CARIBOU TERMINUS PLANT
CARIBOU
AROOSTOOK RIVER
N
AROOSTOOK RIVER TREATMENT TRANSPORT SYSTEM
3000 0 3000 6000
FEET
MATCH LINE ---- SEE SOUTHERN AREA

FIGURE 12B
INITIALLY PROPOSED TREATMENT - TRANSPORTATION CONFIGURATION
NORTHERN AREA

The Washburn studies also included transportation of the wastewaters to Presque Isle by both gravity flow, and pumped means. The costs required to accomplish the waste transfer by both means, however, greatly exceeded the cost of providing a treatment facility at Washburn. As concluded in Section VII, the benefits to be gained by such a transfer are not significant enough to justify the additional costs.

Additional studies were undertaken to determine the feasibility of transferring the waste activated sludge, from the Washburn treatment facility, to the head end of the treatment-transport system. As this sludge would amount to about 7,000 pounds per day, it was felt that this could act as a seed sludge for the treatment-transport system. To perform this function, however, the sludge would have to be kept in an aerobic condition from Washburn to Presque Isle, a distance of approximately eleven (11) miles.

Based on published data, [6,9] a sludge of this nature would exert an oxygen demand of about 40 mg/l. If the solids were pumped at a concentration of 1.5%, the resulting pumping rate would be 35 gallons per minute (gpm). With a four (4) inch force main, 18 hours would be required to transfer the solids from Washburn to Presque Isle. With a six (6) inch force main, the resulting time would be 41 hours.

A projection of the time of oxygen depletion, for the waste activated sludge, was determined by:[38]

$$\frac{C_{min}}{C_o} = e^{-kt} \tag{1}$$

C_{min} = minimum allowable dissolved oxygen concentration (0.5 mg/l)
C_o = original dissolved oxygen concentration (5.0 mg/l)
k = oxygen demand of the waste activated sludge (40 mg/l)
t = time in minutes

In order to maintain a dissolved oxygen content of 0.5 mg/l in the sludge, it would have to be aerated every 3.5 minutes. With a four (4) inch force main, this would require about 300 reaeration stations, or one every 200 feet. McCabe and Eckenfelder[41] indicate that sludge of this nature could remain in a viable condition for several hours without aeration. If detailed studies substantiate this, then the number of aeration stations could possibly be reduced 12 or 18.

While such a system of sludge transfer would probably be technically feasible, it was not adopted because of the following:

1. More detailed studies would have to be undertaken to determine the extent of sludge re-aeration.
2. The capital and operating costs can not be justified in relation to the benefits gained.

The treatment-transport system configuration for detailed study, as shown on Figures 13A and 13B consisted of:

1. Pumping the Easton area waste flows to the Aroostook River Basin.
2. Combining the Presque Isle area, and Easton area waste flows for treatment while in transit to Caribou.
3. Terminal treatment facilities at Caribou for handling the Caribou area waste flows and the discharge from the treatment-transport system.

HYDRAULIC AND ORGANIC LOAD CONDITIONS - For analysis of the treatment-transport system, three loading conditions were assigned. The makeup of these loadings is summarized in Table 41.

TABLE 41

DESIGN LOADINGS - TREATMENT-TRANSPORT SYSTEM

	Load Conditions					
	High		Medium		Low	
Contributor	Flow	BOD	Flow	BOD	Flow	BOD
Presque Isle Domestic	1.4 mgd	3000 lb/d	1.4 mgd	3000 lb/d	1.4 mgd	3000 lb/d
Potato Service Inc	3.1	48870	2.9	42650	2.0	18350
Easton Municipal	0.2	460	0.2	460	--	--
Vahlsing Inc	2.1	25000	1.7	2100	--	--
Maine Sugar Industries	2.8	16000	--	--	--	--
Totals	9.6	93330	6.2	67020	3.4	21350

The high loading condition assumes full participation by all municipal and industrial entities. The industrial loads represent the highest levels, as previously presented, while the domestic loads are based on a 20 year projection. The medium load condition assumes modest in-plant conservation success at Potato Service, Inc, and Vahlsing, Inc, but omission of Maine Sugar Industries. In addition, the low loading condition assumes no input from the Easton area for legal, or other administrative reasons. It is emphasized that the above loadings represent a range for analysis, and in actuality, different combinations could realize almost any hydraulic or organic load within this range. Prior to final design, all participants and their loadings must be accurately determined.

HYDRAULIC STUDIES - The topography along the Aroostook River drops from elevation 445. at Potato Services, Inc to elevation 390. at the Caribou treatment facility. Several alternative channel routes were investigated, all

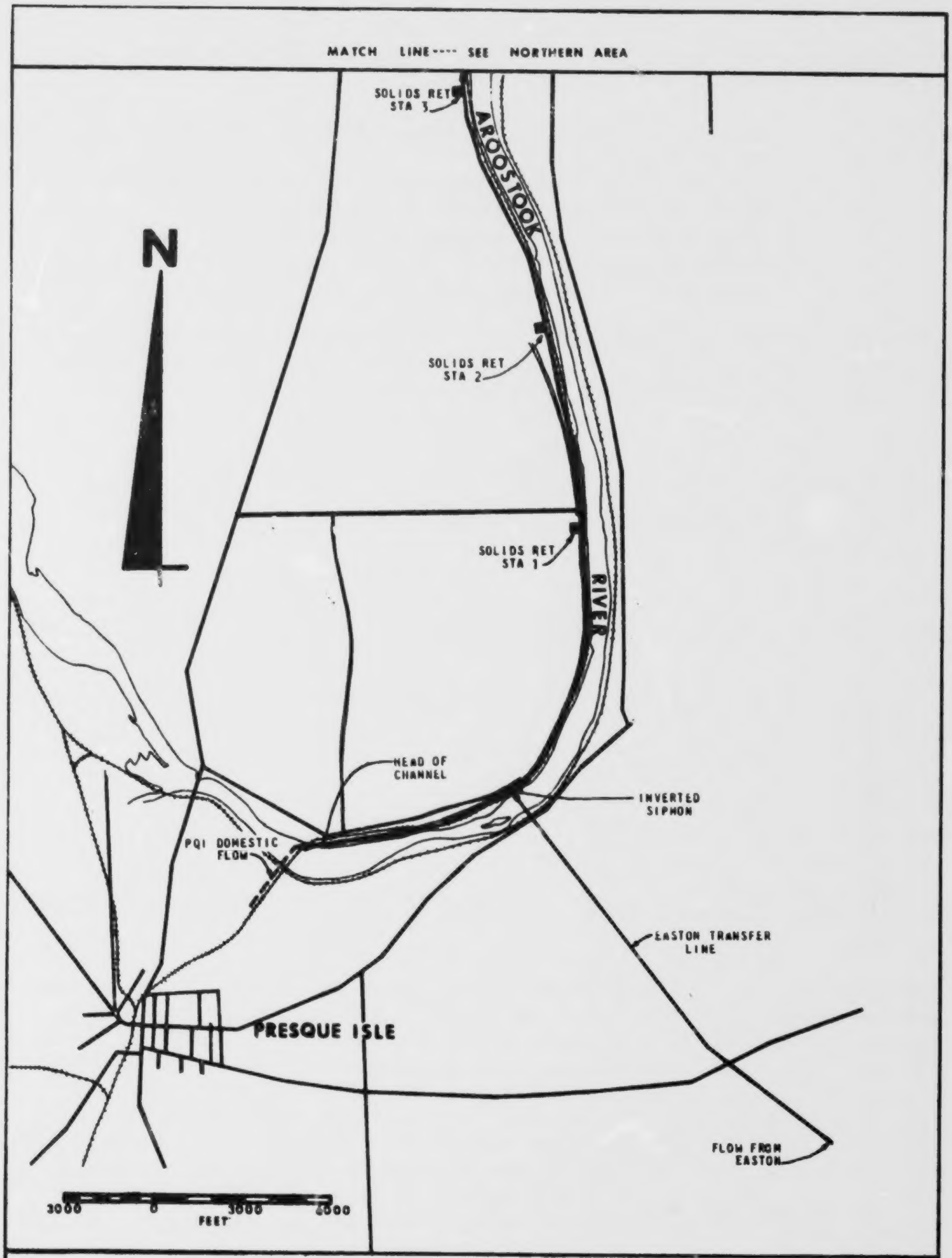

MATCH LINE ---- SEE NORTHERN AREA
SOLIDS RET STA 3
AROOSTOOK
N
SOLIDS RET STA 2
SOLIDS RET STA 1
RIVER
HEAD OF CHANNEL
INVERTED SIPHON
PQI DOMESTIC FLOW
EASTON TRANSFER LINE
PRESQUE ISLE
FLOW FROM EASTON
3000
0
3000
6000
FEET

FIGURE 13A
FINAL TREATMENT - TRANSPORTATION CONFIGURATION
SOUTHERN AREA

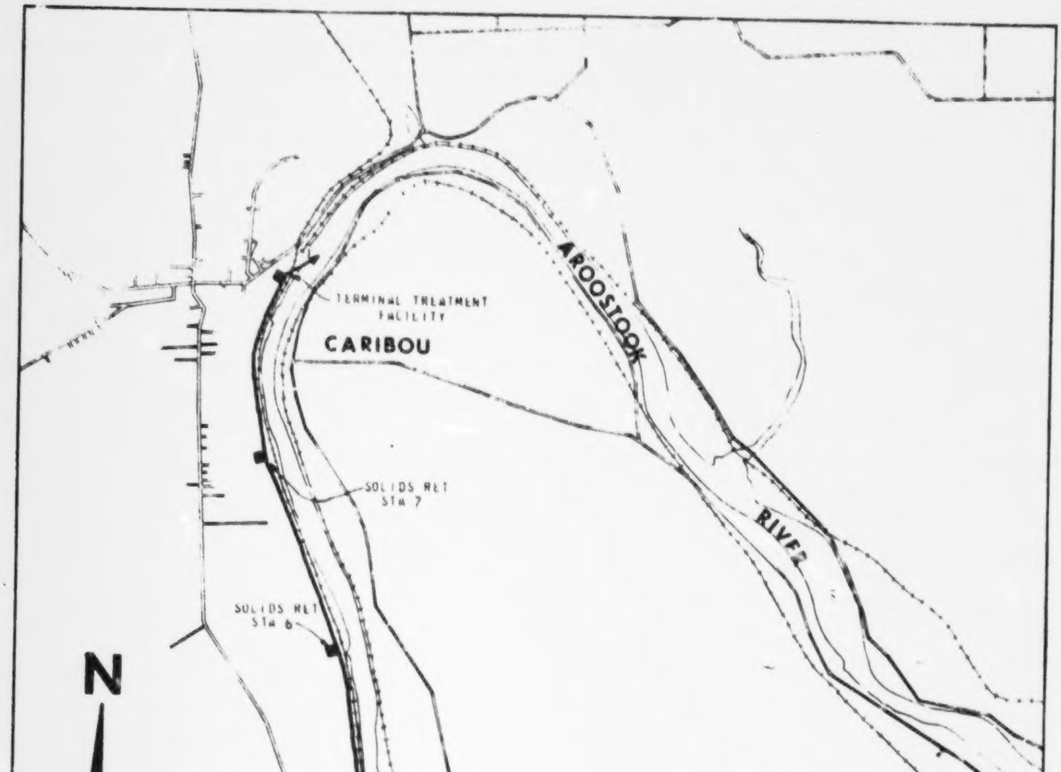

TERMINAL TREATMENT FACILITY
CARIBOU
AROOSTOOK
RIVER
SOLIDS RET STA 7
SOLIDS RET STA 6
SOLIDS RET STA 5
SOLIDS RET STA 4
N
3000
0
3000
6000
FEET
MATCH LINE---- SEE SOUTHERN AREA

FIGURE 13B
FINAL TREATMENT - TRANSPORTATION CONFIGURATION
NORTHERN AREA

of which allowed gravity flow from Potato Services, Inc to the Caribou treatment facility. Waste flows from the Presque Isle and the Easton areas would be pumped to the Potato Services site. The proposed route runs primarily adjacent to the Bangor and Aroostook RR right-of-way. The other alternatives paralleling this route were discarded because of excessive earth work, unfavorable construction conditions near the river bank, possibility of flooding and washout from excessive river flows, and numerous crossings of the railroad right-of-way.

Several combinations of channel slope were studied for the proposed reach. The combination which provided for acceptable flow conditions, with a minimum of earth work and rock excavation, was a reach of slope at .0008 followed by a reach at .0004. This slope combination would create the least interference with existing drainage patterns. Several streams, the largest of which is the Caribou Stream, must be crossed with pipe bridges or inverted siphons. Drainage structures to accommodate these and other flows were considered and must be included in any design of a channel paralleling the river.

In order to maintain uniformity within the treatment process through a widely varying flow regime, it is desirable to provide a channel section which will maintain a relatively constant velocity, and transit time. A study was undertaken to determine the most efficient channel configuration for both hydraulic and treatment considerations. Those shapes analysed included rectangular, trapezoidal, circular, elliptical and parabolic. The hydraulics of these shapes were investigated for flow ranges from 1.0 through 23.0 cfs. Table 42 presents the velocities and water depths for slopes of .0004 and .0008 for the shapes investigated.

The circular, elliptical and trapezoidal channel slopes were discarded because of inadequate depth for proper aeration, and because of the relatively large variation in velocities between high and low flow conditions. The rectangular channel, especially that with a 2.0 foot base width, provided for a reasonably constant velocity over the flow range anticiapted. For low flow, however, the depth of flow is insufficient for proper aeration of the waste.

Several parabolic sections were investigated. These channel sections were generated by the following relationships: $y = x^2$, $y = 2x^2$ and $y = 5x^2$. The $y = 5x^2$ section provided for the most constant velocity with varying flow rates, and also maintained sufficient depth for aeration purposes. Preliminary biologic studies indicated that an extensive biologic solids return system would probably be required for the channel treatment system. Solids return would be accomplished by pumping through return pipes embedded in the concrete section. A typical section through the channel showing dual return piping is shown in Figure 14.

The hydraulics of the channel have been evaluated to determine the flow vs depth and flow vs discharge relationships for solids return rates of 0% to 50% of the average inflow. The average inflow was taken at 9.6 mgd. These flow vs depth relationships for the channel slopes of 0.0004

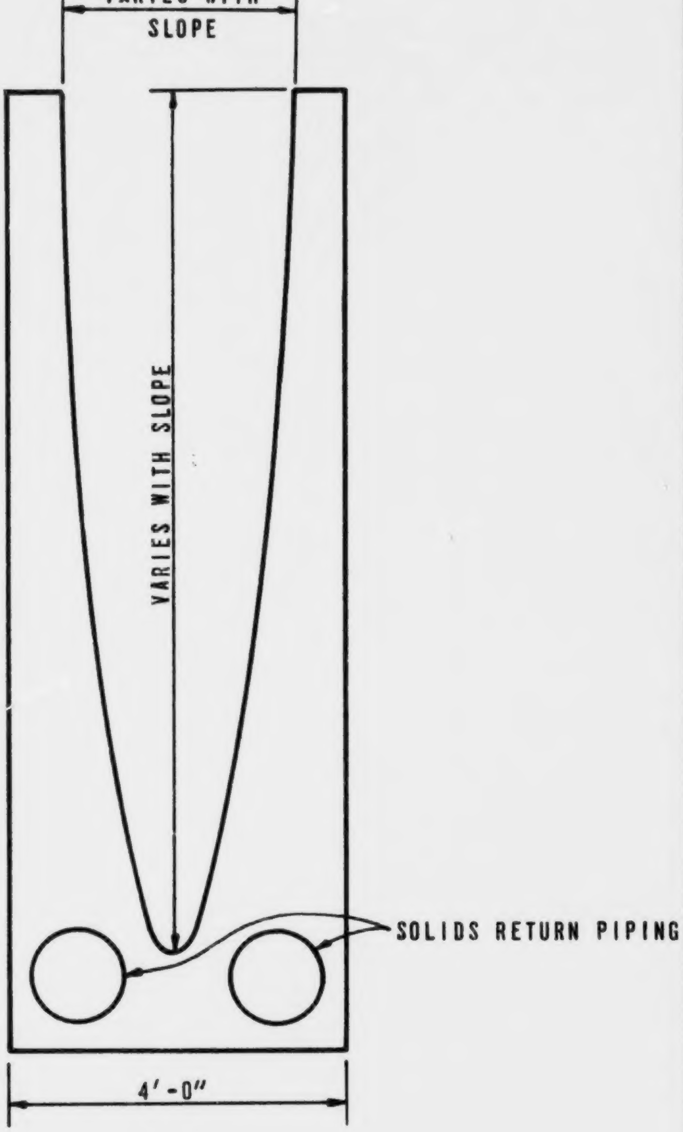

VARIES WITH SLOPE
VARIES WITH SLOPE
SOLIDS RETURN PIPING
4'-0"

FIGURE 14
PARABOLIC CHANNEL

TABLE 42

VELOCITIES AND WATER DEPTHS FOR VARIOUS CHANNEL CONFIGURATIONS

	S = .0004						S = .0008					
	2.0 cfs		9.0 cfs		23.0 cfs		2.0 cfs		9.0 cfs		23.0 cfs	
	Vel fps	Depth ft	Vel fps	Depth ft	Vel fps	Depth ft	Vel fps	Depth ft	Vel fps	Depth ft	Vel fps	Depth ft
Rectangular												
1. base width = 2.0 ft	1.3	0.9	1.6	3.0	2.0	5.9	1.5	0.7	2.1	2.1	2.5	4.5
2. base width = 3.0 ft	1.2	0.6	1.7	1.8	2.1	3.4	1.4	0.5	2.2	1.4	2.8	2.7
Trapezoidal												
1. base width = 2.0 ft 2:1 side slopes	0.9	0.5	1.7	1.3	2.0	2.0	1.2	0.4	2.1	1.1	2.7	1.6
Circular												
42-inch	1.5	0.7	2.0	1.6	2.6	3.5	1.8	0.6	2.7	1.3	3.6	2.6
48-inch	1.4	0.7	2.1	1.5	2.7	2.5	1.8	0.5	2.7	1.2	3.5	2.1
Parabolic												
1. $y - 5x^2$	1.4	1.9	1.7	4.2	2.0	7.0	1.7	1.6	2.3	3.7	2.6	6.0

and 0.0008 are illustrated in Figures 15 and 16. For the initial slope of 0.0004 and a solids return rate of 3.7 cfs, the channel flow depth would range from 4.4 feet with inflow of 2.0 cfs to 8.3 feet with an inflow of 25.0 cfs. For the section at slope 0.0008, the above conditions would maintain flow depths of 3.6 feet and 7.1 feet, respectively. As can be seen in Figures 15 and 16, for the high solids return rates, there is less depth variation with inflow.

Figures 17 and 18 illustrate the change in velocity with inflow for the two design slopes. With a return rate of 3.7 cfs, the velocities would range from 1.72 fps at an inflow of 2.0 cfs to 2.20 fps at an inflow of 25.0 cfs with the 0.0004 channel slopes. For the 0.0008 slope, channel corresponding velocity values would be 2.30 fps and 2.94 fps, respectively.

The low flows occur during the summer months when the processing plants are shut down. With only the domestic flow from Presque Isle of 1.4 mgd (2.26 cfs) and a solids return rate of approximately 1.0 cfs, the velocities for the 0.0004 and 0.0008 channel slopes would be 1.5 fps and 2.0 fps, respectively. The respective flow depths would be 2.1 feet and 2.0 feet. The 1.5 fps velocity is just adequate to maintain solids in solution; the 2.0 fps velocity in the longer, steeper section is completely adequate.

Velocities may be converted to time of transit from the vicinity of Potato Services to the vicinity of the Caribou treatment facility for the proposed channel route and slopes by utilizing Figure 19. With a solids return rate of 3.7 cfs the residence time of flow will range from 8.6 hours at an inflow rate of 25 cfs to about 11.0 hours at an inflow rate of 2.0 cfs. Within the design flow rates the transit time will vary from approximately 7 to 9 hours.

PROVISION FOR FUTURE EXPANSION - The design for the channel section allows for industrial load levels at present high rates and a 20 year projection for domestic flow, coupled with provisions for a solids return rate of 4.8 mgd. The channel invert would be set so that two extra feet could be added to the top of the channel section. These two feet would allow for increased flows of 15.0 cfs and 18.0 cfs for the 0.0004 and 0.0008 channel slopes, respectively. These flows are comparable to increasing the capacity of the conduit to allow for discharges and 50% solids recirculation approximating those flows from both the Potato Services, Inc and Maine Sugar Industries during high flow periods. This provision for future expansion is felt to be adequate for the design life of the channel.

FLOW REDUCTION IN THE AROOSTOOK RIVER - Prior to construction of the conduit, wastewaters are discharged directly into the Aroostook River at the point of use. After construction of the channel the flow of the Aroostook River between Presque Isle and Caribou will be reduced by the inflow to the treatment-transport channel between these two points.

The maximum flow reduction with the system under design loading conditions should be approximately 7.0 cfs occurring at Presque Isle; consisting of the Presque Isle municipal and Potato Services, Inc wastewaters.

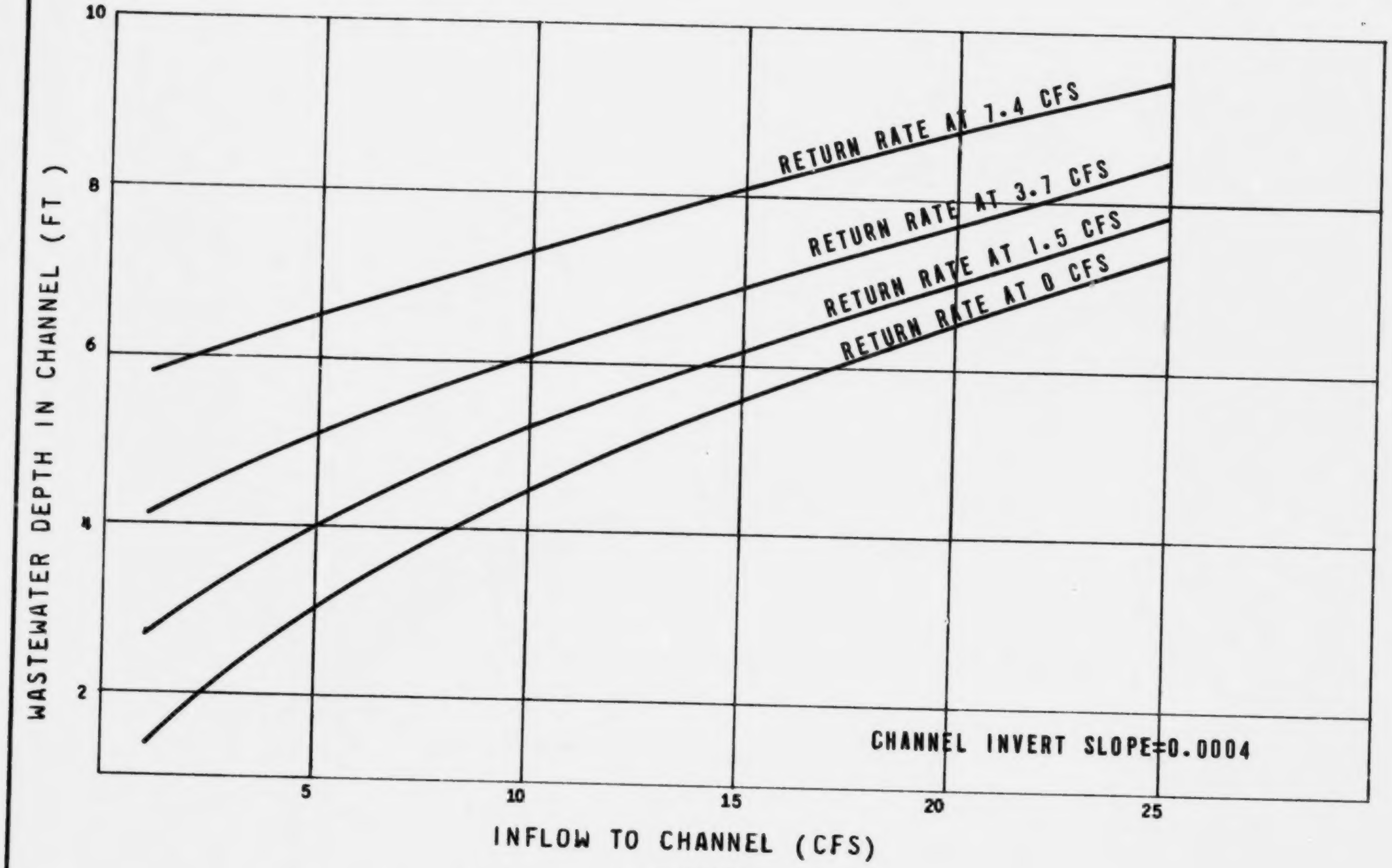


FIGURE 15
WASTEWATER DEPTH IN CHANNEL VS INFLOW

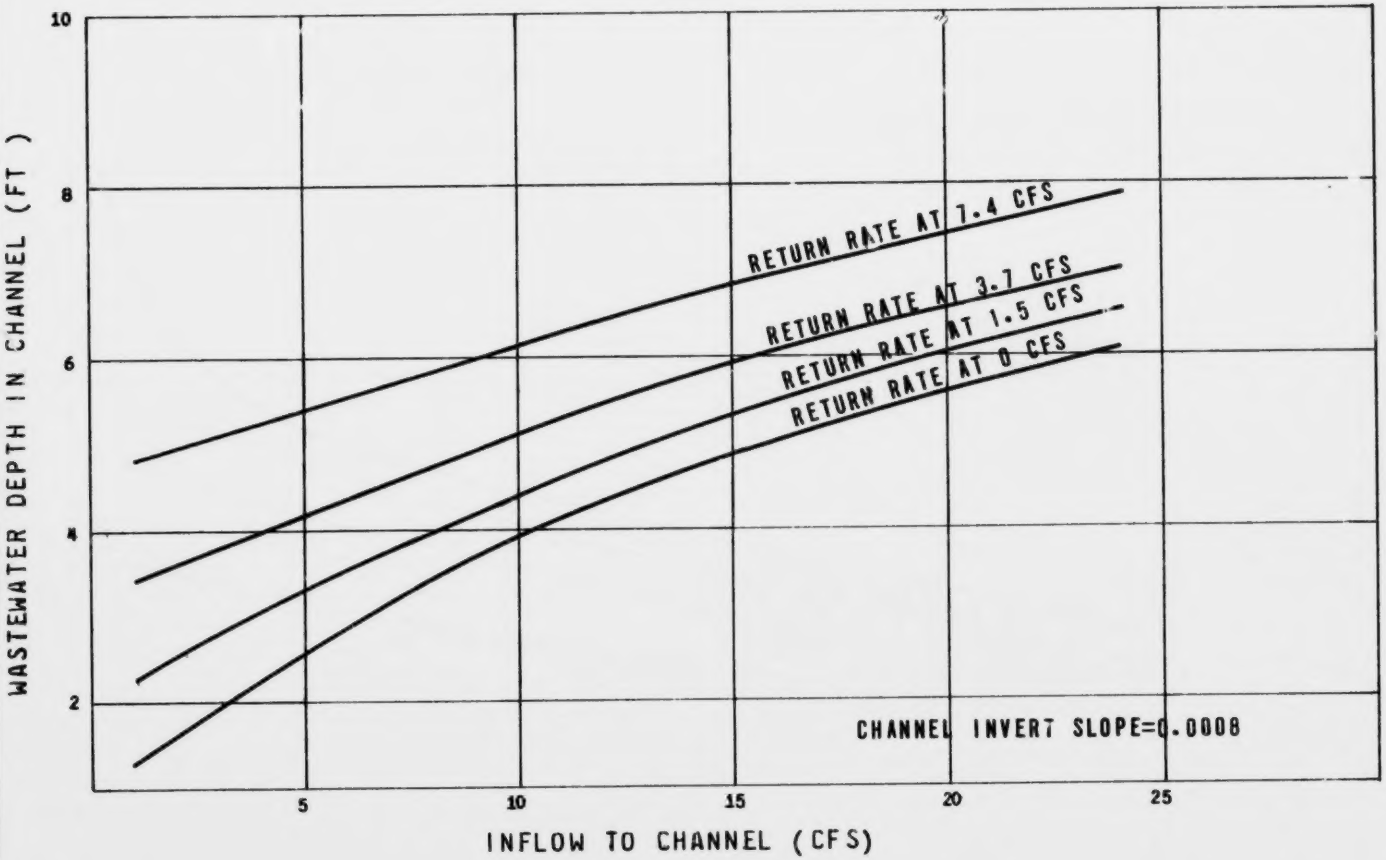


FIGURE 16
WASTEWATER DEPTH VS INFLOW

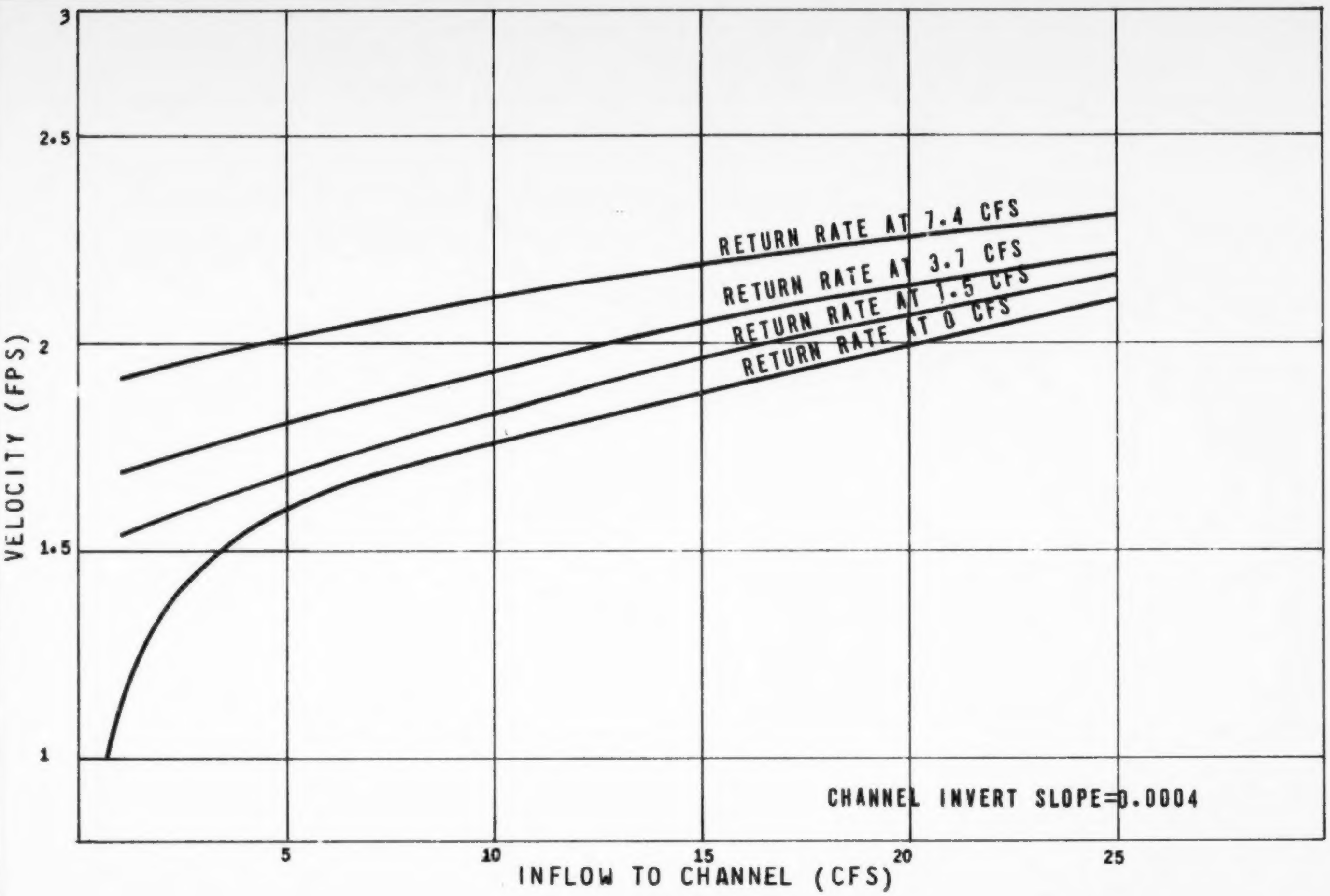


FIGURE 17
VELOCITY VS INFLOW

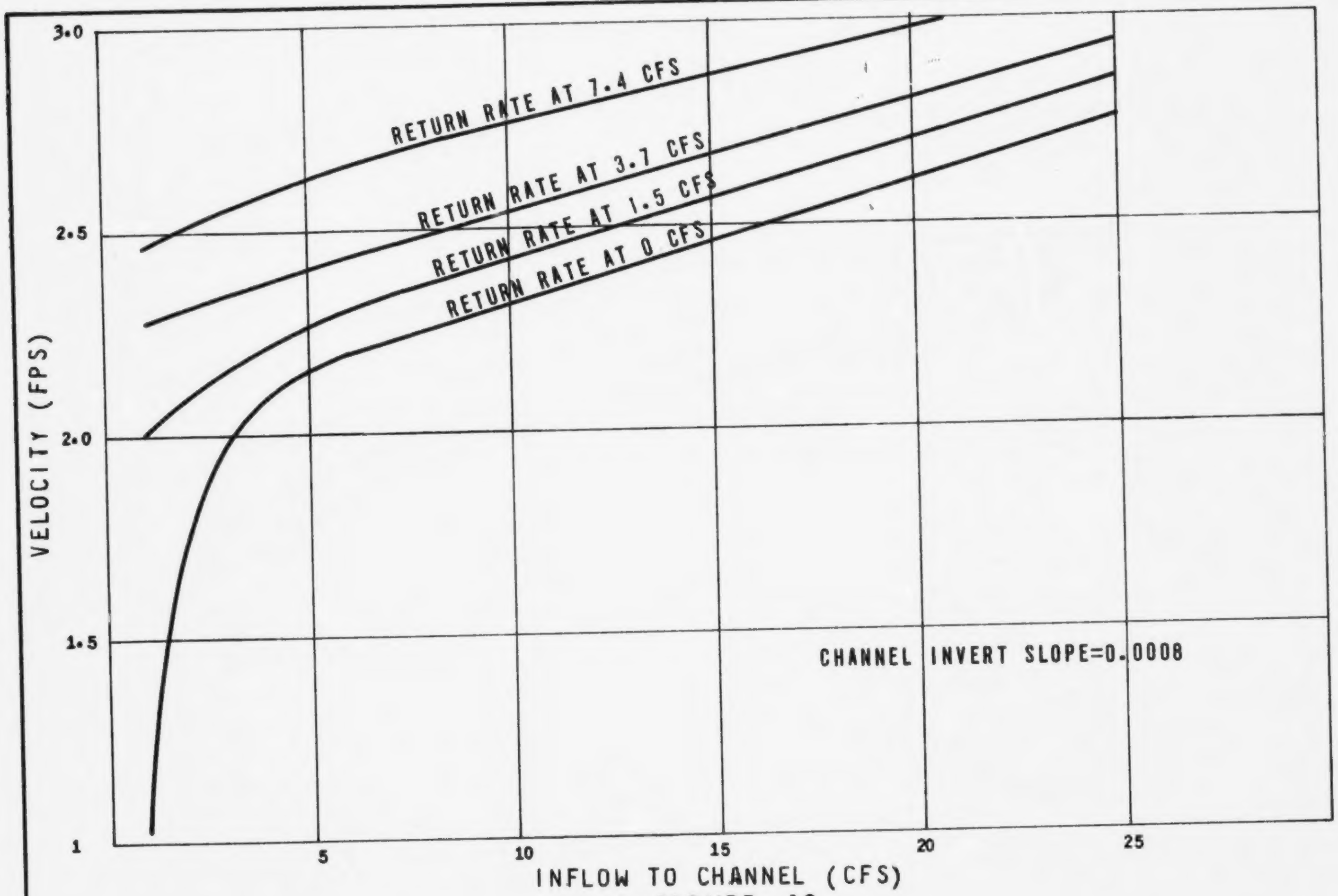


FIGURE 18
VELOCITY VS INFLOW

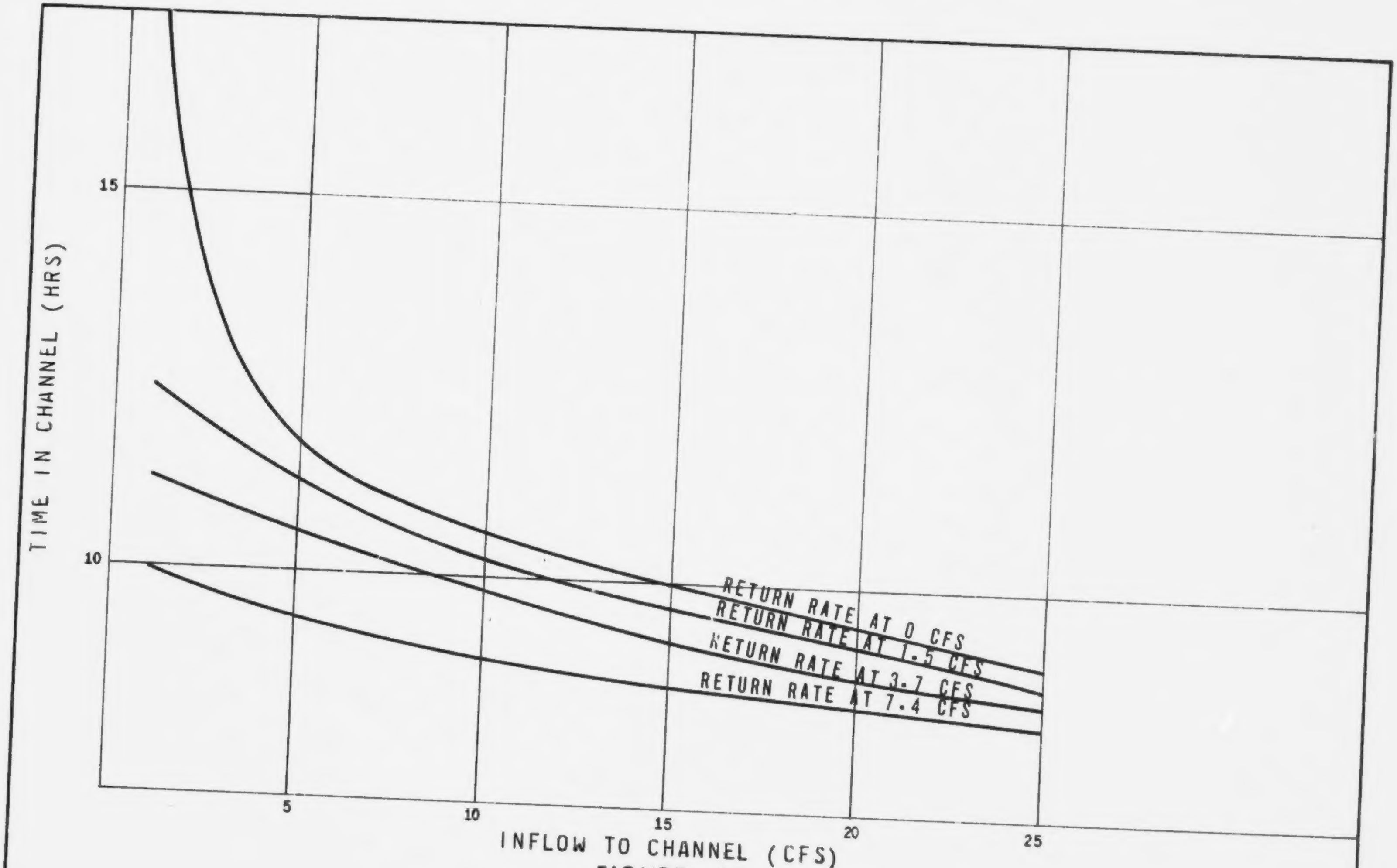


FIGURE 19
TIME IN CHANNEL VS INFLOW

The 10-year seven day and one day minimum flows at Presque Isle are 125. cfs and 110. cfs, respectively. The 2-year seven-day and one day minimum flows are 310. cfs and 240. cfs. A 7.0 cfs flow reduction amounts to only approximately 6% for the 10-year return interval. This minor reduction is completely acceptable and should have no deleterious results.

AERATION STUDIES - To achieve the required aeration of the biological mass in the treatment conduit, oxygen must be introduced to the system. Theoretically, the introduction of oxygen may be accomplished by several methods. These methods are analyzed in the following paragraphs.

Aeration may be accomplished with devices inserted into the flow to increase the natural turbulance in the channel. These devices may be baffles, cascades or increased roughness of the channel walls such as rip-rap. The use of this type of aeration procedure is desirable because no energy input other than gravity is required. Unfortunately, the channel slope from Presque Isle to Caribou is too flat to allow for this type of aeration. As presented in Table 42, the flow velocity for the various channel shapes borders on the minimum, which will prevent deposition of solids. Increasing the slope is not possible, and introducing increased roughness at the proposed slope would reduce the channel velocities below acceptable values for transporting solids.

The use of periodic aeration basins along the channel route was also considered. The head loss through these units, however, combined with that through the solids concentrators was too great. In order to allow for this head loss, the channel slope would have had to be flattened with attendant low velocity problems.

The introduction of pure oxygen into covered oxidation chambers along the channel route was investigated and discarded because the number of installations required proved this method to be uneconomical. Also the head loss through each of these chambers would have been excessive.

The use of diffused air introduced continuously along the channel was investigated and concluded to provide the best means of achieving the required oxygen transfer to maintain aerobic biological activity. This process is discussed in detail in the following paragraphs.

An analysis of the oxygen transfer from bubble aeration performance is presented below. Bubble aeration was calculated to be the prime transfer mechanism based on the studies by Downing[39], which stated that surface aeration transfer varies from 4.45% of the total transfer at a 4.0 foot depth to only 2.21% of total oxygen transfer at a 12.0 foot depth. Neglecting surface aeration in the analysis provides for a degree of conservativeness. Surface areas in the channel are presented in Figures 20 and 21 for the proposed slopes. It can be seen from these figures that the narrow channel does not provide appreciable surface area for any depth.

Much of the procedure and parameters for the required spacing of diffusers was taken from Ecenfelder and O'Conner[38]. The oxygen absorbtion number N is determined by the following relationships:

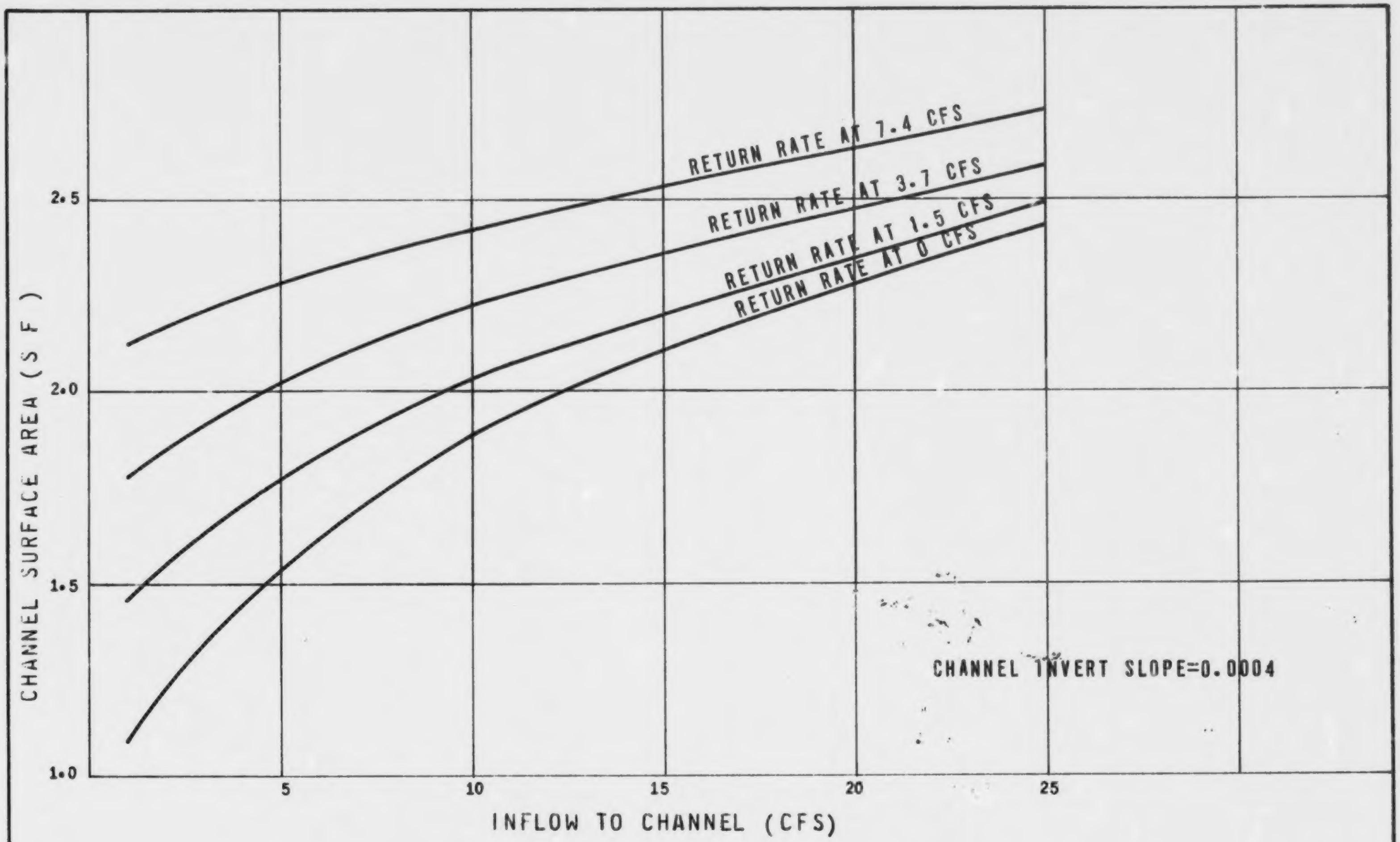


FIGURE 20
CHANNEL SURFACE AREA VS INFLOW

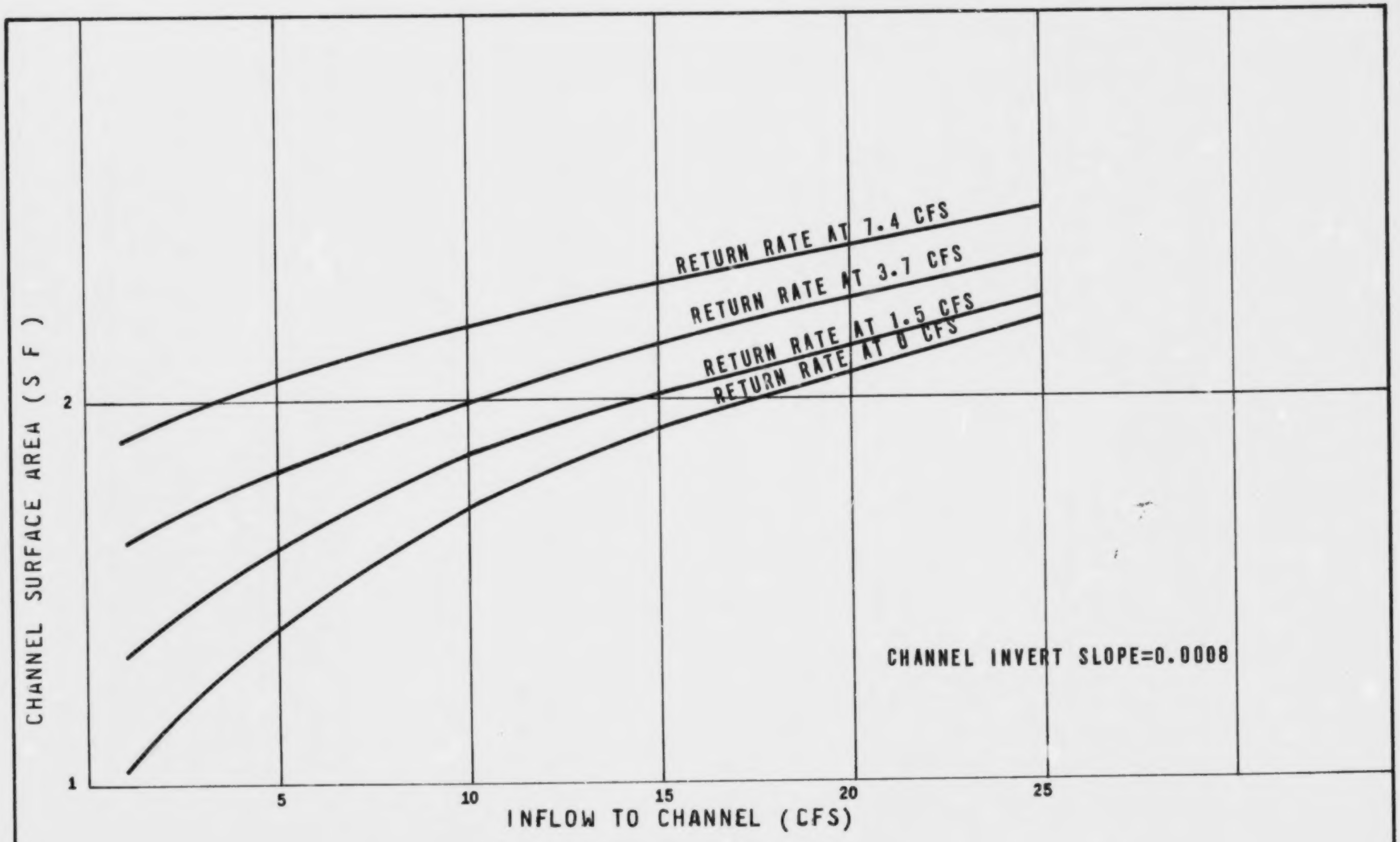


FIGURE 21
CHANNEL SURFACE AREA VS INFLOW

$$N = \frac{VK_La}{G_s^{(1-n)}\ h^{(1-g)}} \qquad (2)$$

Where N = absorbtion number for aeration diffusers
V = Volume of flow being aerated
K_La = reaeration coefficient
G_s = air flow per diffuser (SCFM)
(1-n) = gas rate exponent
(1-g) = depth parameter

$$K_La = \frac{r_r}{(C_s - C_L)}\ \frac{1}{\alpha}\ \frac{1}{1.219} \qquad (3)$$

Where: r_r = oxygen uptake rate (mg/l/hr)
C_s = oxygen saturation (mg/l)
C_L = concentration of dissolved oxygen (mg/l)
α = ratio of K_La in wastewater to K_La in water
$\frac{1}{1.219}$ = temperature correction for temperature = 30°C

Substituting equation (3) into equation (2) and rearranging yields:

$$Vr_r = 1.219\alpha N\ (C_s - C_L)\ h^{(1-g)}G_s^{(1-n)} \qquad (4)$$

If V is taken as the cross-section area (A) of the channel times the distance between diffusers (d) and substituted into equation (3), the final relationship developed is:

$$\text{dist x } r_r = \frac{1.219}{A}\alpha\ N(C_s - C_L)\ h^{(1-g)}G_s^{(1-n)} \qquad (5)$$

This relationship provides a means for determining the distance between diffusers for various depths of flow and for various oxygen uptake rates. For the purpose of this analysis, values for several of these parameters were taken from the literature in order to provide a means for determining the number and spacing of diffusers.

While those precise values to be utilized for a final design should be determined from pilot plant studies, the following values are believed to realistically represent the waste conditions to be expected. The design operating temperature was taken at 30°C, oxygen transfer being more efficient at lower temperatures. The following key oxygen transfer design criteria were adopted:

C_L range 0.0 - 2.0 mg/1
α = 0.80
Oxygen transfer 10% from bubbles to wastewater
Saturation DO (C_s) = 7.5 mg/1
Air flow per diffuser - 10. SCFM
N = 30
1-n = 1.0
1-g = 0.78

Oxygen transfer rates were computed assuming a sparjer type diffuser and were varied with depth of flow in the channel. The oxygen transfer along the channel would be varied to meet oxygen demand by adjusting the spacing of the sparjers, as required. Figure 22 presents the results of the analysis in a plot of channel depth versus a distance-oxygen uptake factor expressed in ft x mg/1/hr. The figure also indicates curves with mixed liquor dissolved oxygen concentrations ranging from 0.0 to 2.0 mg/1. From this data, and the oxygen demand studies, it is possible to select diffuser spacing along the channel and to compute air flow requirements.

Oxygen demand analysis suggest that peak oxygen demand could range up to about 70. mg/1/hr with maximum design loading, dropping to a low of about 15. mg/1/hr at the end of the channel with minimum loading conditions. As an example of utilizing Figure 22, assume the depth of flow is 4.0 feet and C_L = 1.0 mg/1. Dist x r_r is taken from the curve to be 1210 feet mg/1/hr. If the uptake rate is 60. mg/1/hr the distance between diffusers would be 20.2 feet.

BIOLOGICAL STUDIES - As previously stated, the original program included a pilot plant study for determination of the biological design parameters. This phase of the program, however, was deleted. As a result, the design parameters had to be established from literature.

Three variations of a biological system for the treatment-transport channel were considered, as follows:

1. Aeration in channel with no solids return and no insertion of active biologic mass at the head of the channel.
2. Insertion of an activated biologic mass in the head of the channel, with aeration and no solids return.
3. Aeration in channel with extensive solids return system.

Alternate 1 above provides simply for aerating the waste flows while in transit from Presque Isle to Caribou. While this would be technically possible, the only benefit to be gained is maintaining the waste flows in a "fresh" condition. With such a system, little BOD reduction would be achieved due to the inability of the system to generate an active biological growth within the time of flow (7 to 11 hours, depending on flow rate). In an actual system, however, some BOD removal could be achieved by the virtue of growths along the channel surfaces acting as a seed. The extent of such growths, resulting BOD removal, and oxygen demand would have to be determined through detailed pilot plant studies.

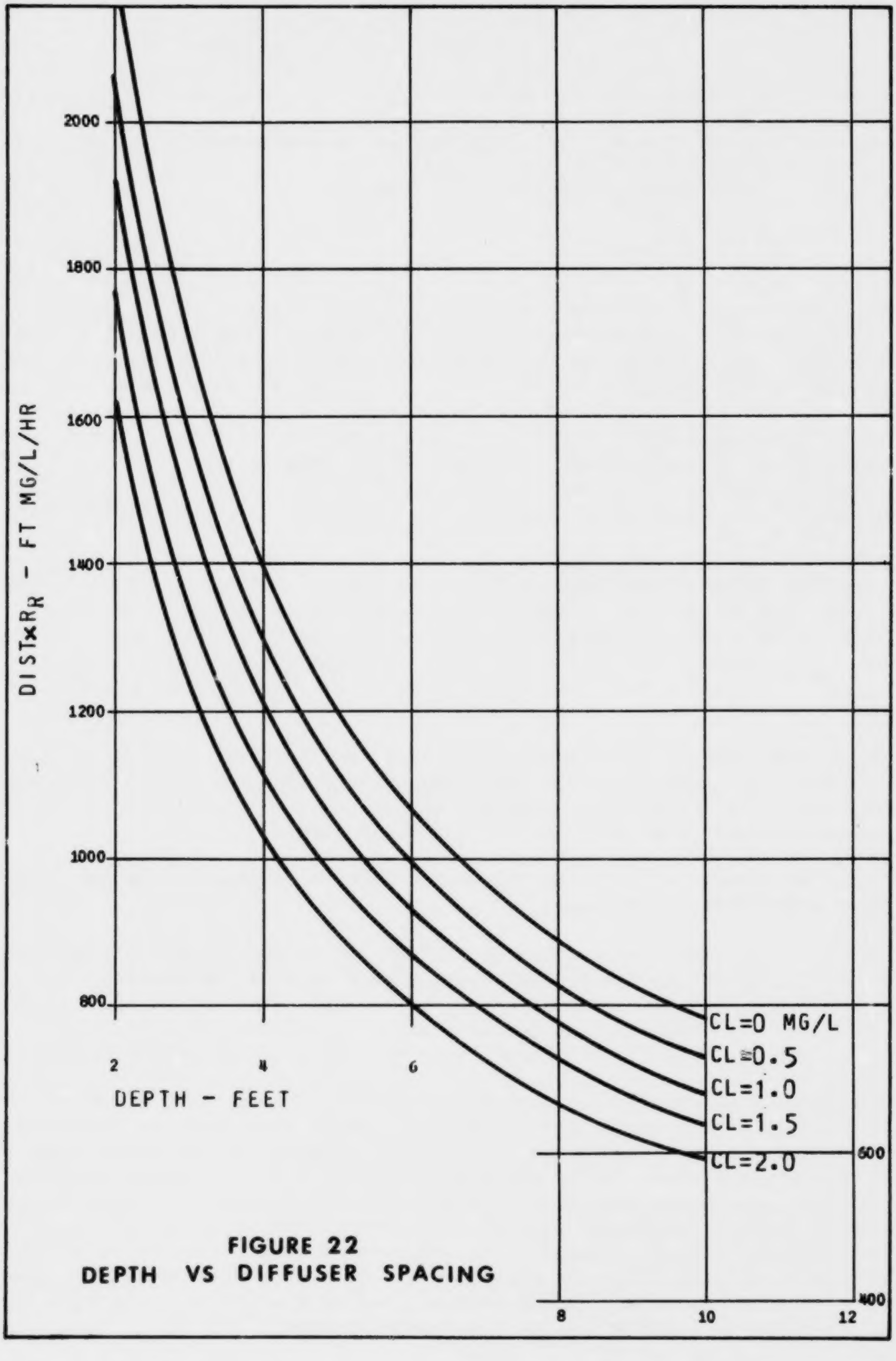


FIGURE 22
DEPTH VS DIFFUSER SPACING

Alternate 2 above requires the addition of an active biological mass at the head end of the channel. After several rough approximations it was decided to evaluate this system by adding sufficient solids at the head end of the channel to increase the MLVSS by 500 mg/l. For the three design loadings, the following quantities of solids would be required:

Load Cond	Flow Rate	#Solids Required
Maximum	9.6 MGD	38,400 #/day
Medium	6.2 MGD	25,800 #/day
Minimum	3.4 MGD	14,200 #/day

A schematic diagram of the system configuration for this alternate is shown in Figure 23.

The establishment of the various design parameters and evaluation of the above system were accomplished by the following formulations:

Oxygen Requirement: (Eckenfelder & O'Connor[38])

$$\#O_2/\text{day} = (a')(\#BOD_{REMO}/\text{day}) + (b')(\#MLVSS) \qquad (6)$$

Where:
$a' = 0.48$
$b' = 0.03$

Upon determination of the oxygen requirements by equation (6) the total air requirement was determined from the curves previously presented for channel aeration. The use of the channel aeration curves allows determination of the number of air diffusers, blower size, and air header size.

BOD removals along the various reaches of the channel were determined by:[38]

$$\text{BOD Removal} = (100)\left(\frac{k_s S_a t}{1+k_s S_a t}\right) \qquad (7)$$

Where k_s = 0.00047 (sludge growth factor)[9]
S_a = MLVSS concentration
t = time of aeration

Solids production along the various reaches of the channel were determined by:[38]

$$\#\text{Solids Growth/Day} = (a)(\#BOD_{REMO}) - (b)(MLVSS) \qquad (8)$$

Where $a = 0.65$
$b = 0.05$

Tables 43, 44 and 45 presents the data concerning solids production and BOD removals along the channel reach for the minimum, medium, and maximum load conditions. These tables show projected BOD removals for the Alternate 2 system in the order of 70 to 80%.

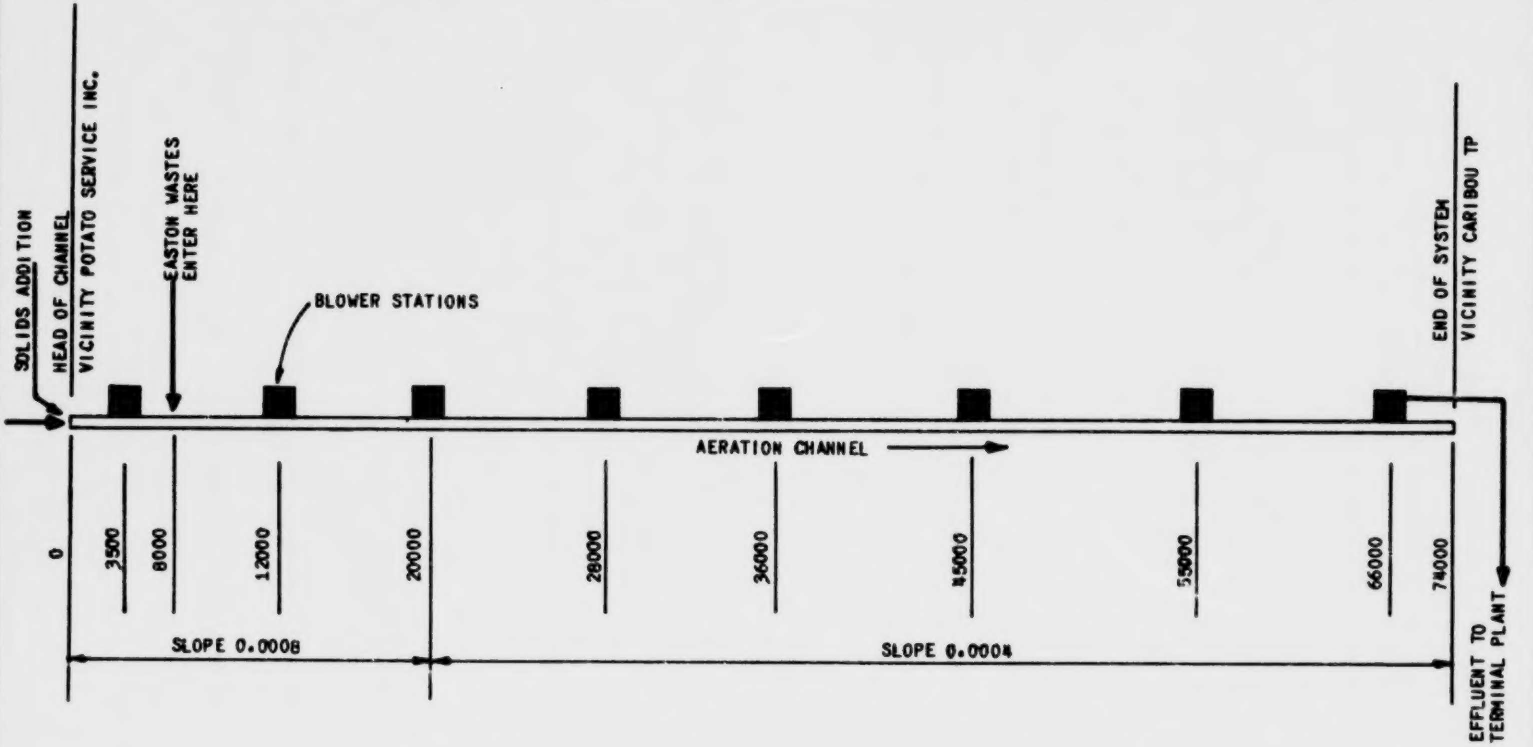

SOLIDS ADDITION
HEAD OF CHANNEL
VICINITY POTATO SERVICE INC.
EASTON WASTES ENTER HERE
BLOWER STATIONS
END OF SYSTEM
VICINITY CARIBOU TP
AERATION CHANNEL
0
3500
8000
12000
20000
28000
36000
45000
55000
66000
74000
SLOPE 0.0008
SLOPE 0.0004
EFFLUENT TO TERMINAL PLANT

FIGURE 23
SCHEMATIC DIAGRAM
TREATMENT - TRANSPORT SYSTEM - ALTERNATE 2

TABLE 43

CHANNEL BIOLOGICAL CHARACTERISTICS

MINIMUM LOAD

ALTERNATE 2

Channel Reach	Time (hrs)	Initial Solids Conc (mg/1)	Vol Solids Prod (lbs)	New Solids Conc (mg/1)	Initial BOD Conc (mg/1)	lb BOD Removed	New BOD Conc (mg/1)	%BOD Removed	lb BOD / 1000 cf	lb BOD / lb MLSS
0-20000	2.57	750	6,200	968	755	9,600	415	45	--	--
0-72000	11.57	750	10,100	1,110	755	16,000	190	75	120	1.84

Hydraulic and Organic Loading

	Flow(MGD)	lb BOD
Presque Isle	1.4	3,000
Potato Service	2.0	18,350
Easton Municipal	0	0
Vahlsing, Inc	0	0
MSI	0	0

TABLE 44

CHANNEL BIOLOGICAL CHARACTERISTICS

MEDIUM LOAD

ALTERNATE 2

Channel Reach (ft)	Time (hrs)	Initial Solids Conc (mg/1)	Vol Solids Prod (lbs)	New Solids Conc (mg/1)	Initial BOD Conc (mg/1)	1b BOD Removed	New BOD Conc (mg/1)	%BOD Removed	1b BOD / 1000cf	1b BOD / 1b MLSS
0-7000	0.85	750	5,865	910	1,270	9,100	1,020	20	2,180	42.0
AT 7000 FT EASTON PORTION ADDED - FIGURES AT 7000 REFLECT NEW CONC AFTER MIXING										
7000-2000	1.57	705	11,000	918	1,120	17,400	785	30		
7000-32000	3.65	705	18,000	1,053	1,120	29,000	560	50		
7000-60000	8.52	705	25,000	1,189	1,120	40,500	336	70		
7000-72000	10.61	705	27,000	1,280	1,120	44,500	253	77	167	2.8

Hydraulic and Organic Loadings

	Flow(MGD)	1b BOD
Presque Isle	1.4	3,000
Potato Service	2.9	42,650
Easton Municipal	0.2	460
Vahlsing, Inc	1.7	21,000
Maine Sugar	0	0

TABLE 45

CHANNEL BIOLOGICAL CHARACTERISTICS

MAXIMUM LOAD

ALTERNATE 2

Channel Reach (ft)	Time (hrs)	Initial Solids Conc (mg/l)	Vol Solids Prod (lbs)	New Solids Conc (mg/l)	Initial BOD Conc (mg/l)	lb BOD Removed	New BOD Conc (mg/l)	%BOD Removed	lb BOD / 1000cf	lb BOD / lb MLSS
0-7000	0.8	750	7,700	928	1,380	10,400	1,100	20	2,240	41
7000-20000	1.47	592	13,400	760	1,040	20,700	780	25		
7000-72000	9.14	592	37,000	1,055	1,040	58,000	312	70	166	2.8

Hydraulic and Organic Loadings

	Flow(MGD)	lb BOD
Presque Isle	1.4	3,000
Potato Service	3.1	48,870
Easton Municipal	0.2	460
Vahlsing, Inc	2.1	25,000
Maine Sugar	2.8	16,000

The BOD concentrations along the channel reach, for the three design conditions, are shown in Figures 24, 25, and 26. As can be seen, the residual BOD concentrations would be in the order of 300 mg/1.

Figures 27, 28, and 29 show the anticipated solids concentration along the channel reach for the three design loadings. Total solids growth in such a system could range from 10,000 to 37,000 pounds per day, depending on actual loading.

The estimated oxygen uptake rates along the channel reach, for the three loading conditions are shown in Figures 30, 31, and 32. The oxygen demand analyses suggest that peak oxygen demands could range up to 160 mg/1/hr at maximum loading, to 15 mg/1/hr at the end of the channel at minimum loading.

Tables 46, 47, and 48 show the aeration requirements in the channel for minimum, medium and maximum load conditions. These tables indicate the oxygen demand, diffuser spacing, number of blowers and blower capacity along the channel.

Tables 49, 50, and 51 show the blower capacity, horsepower, and air pipe sizing for each blower station along the channel system. The total system connected and operating horsepower is also tabulated on these tables.

The overall channel installation will require air piping to be incorporated into the channel section, or laid parallel to the channel.

The biological system for Alternate 3 would most nearly approach a complete mixed activated sludge system for which reasonable design criteria is available. Lacking the opportunity for pilot studies or detailed laboratory analysis, the following general criteria were adopted for design from published data.

Organic Loading = 200 to 400 Lb BOD per 1000 CF
MLSS = 3000 to 4000 mg/1
Detention Time Desired = 8 to 12 hours
BOD to MLSS Ratio = Less than 1.0
MLSS Return Rate = 25 to 50%
K_s = 0.00040
Solids Growth Constants a = 0.65, b = 0.05

A schematic diagram of the Alternate 3 treatment-transport system is shown in Figure 33. Eight solids concentrators and return pump systems are proposed at the locations indicated. These units would consist of an interceptor system in the channel to allow varying portions of the flow to be drawn into a sedimentation chamber which would be equipped with tube settlers and solids collecting mechanisms. Centrigugal pumps would return the solids upstream about 16000 feet for re-entry into the channel. A sketch plan of a typical unit is shown on Figure 34. Blowers for the aeration system would also be located at these units. The entire unit would be covered for cold weather protection. It is noted that the return pipes from two stations overlap to bring the MLSS concentrations up to desired levels.

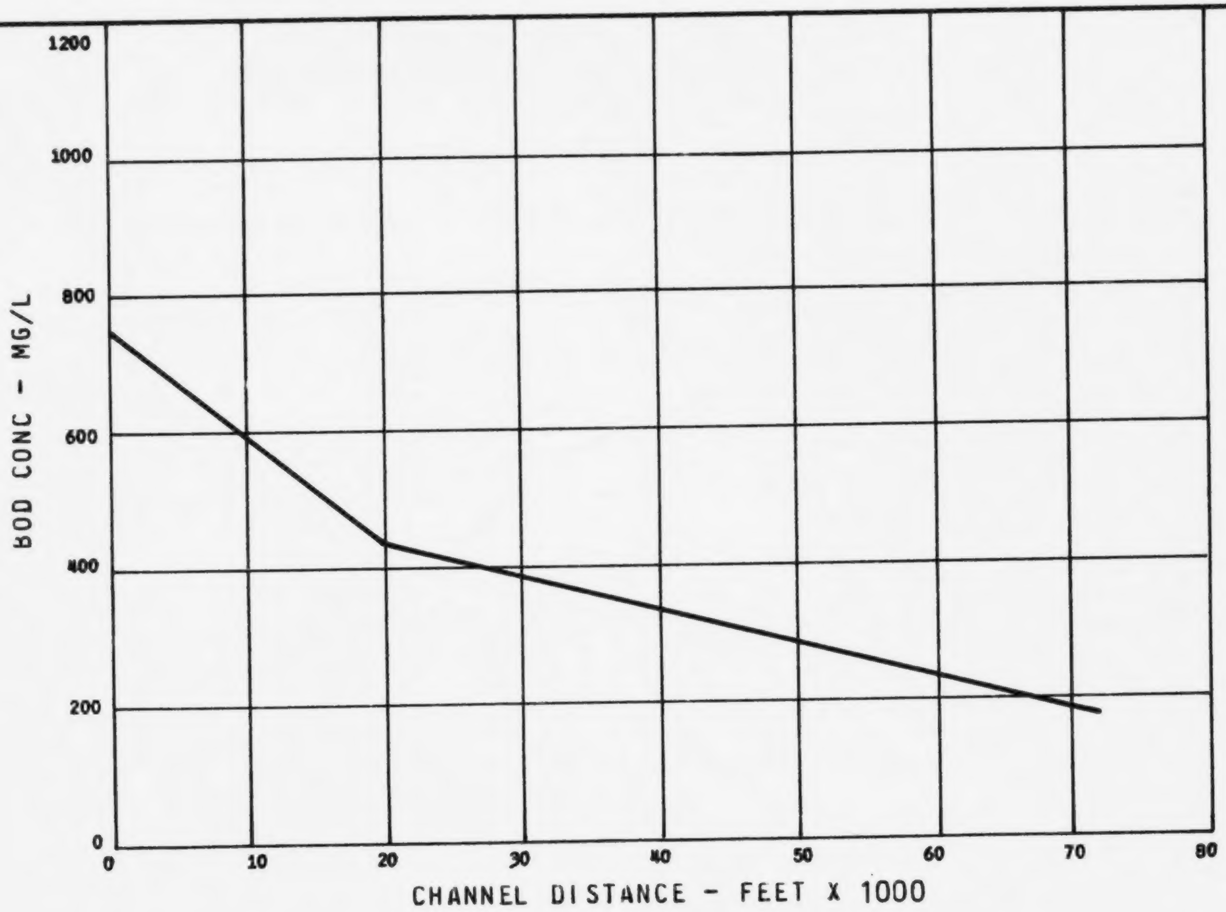


FIGURE 24
BOD CONCENTRATION VS CHANNEL DISTANCE
MINIMUM LOAD - ALTERNATE 2

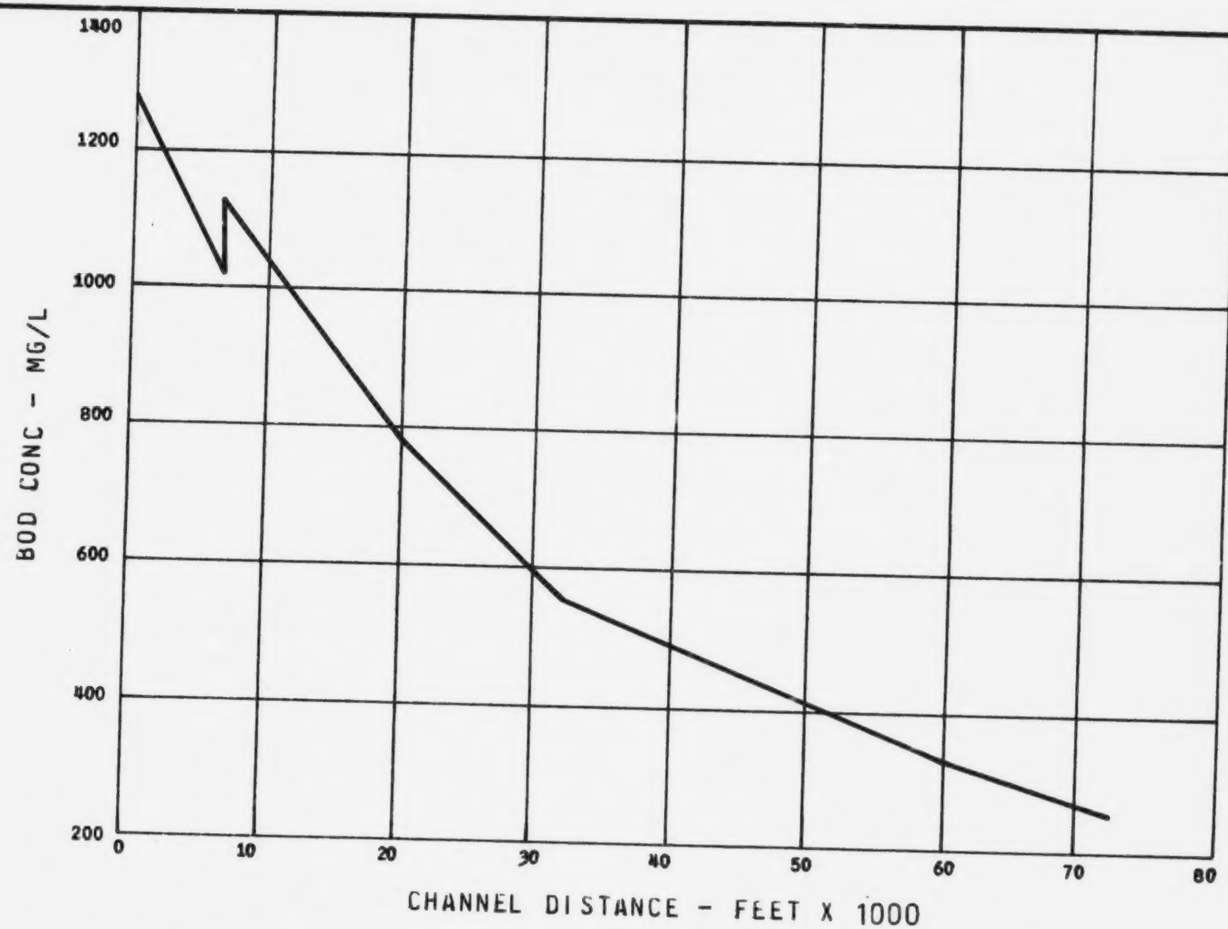


FIGURE 25
BOD CONCENTRATION VS CHANNEL DISTANCE
MEDIUM LOAD ALTERNATE 2

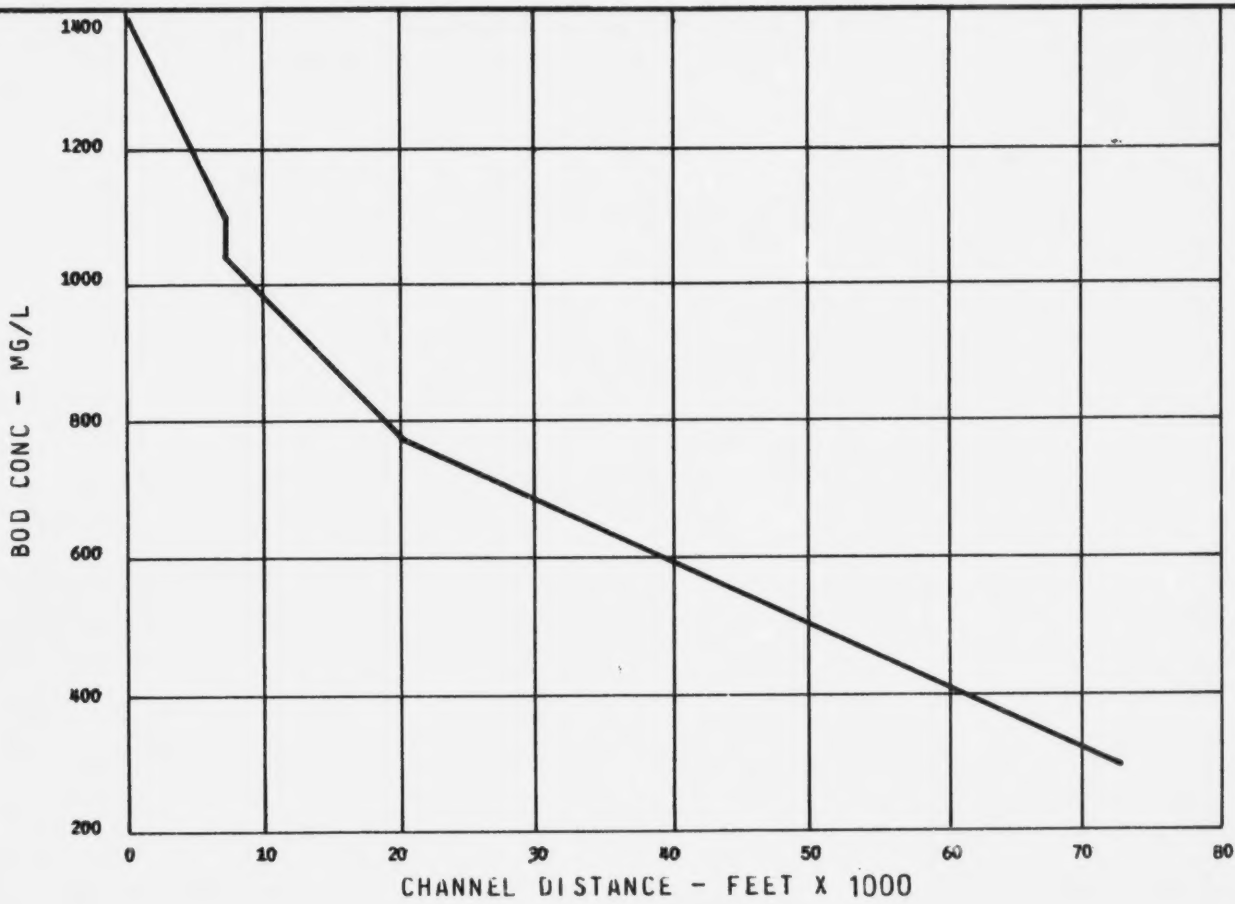


FIGURE 26
BOD CONCENTRATION VS CHANNEL DISTANCE
MAXIMUM LOAD ALTERNATE 2

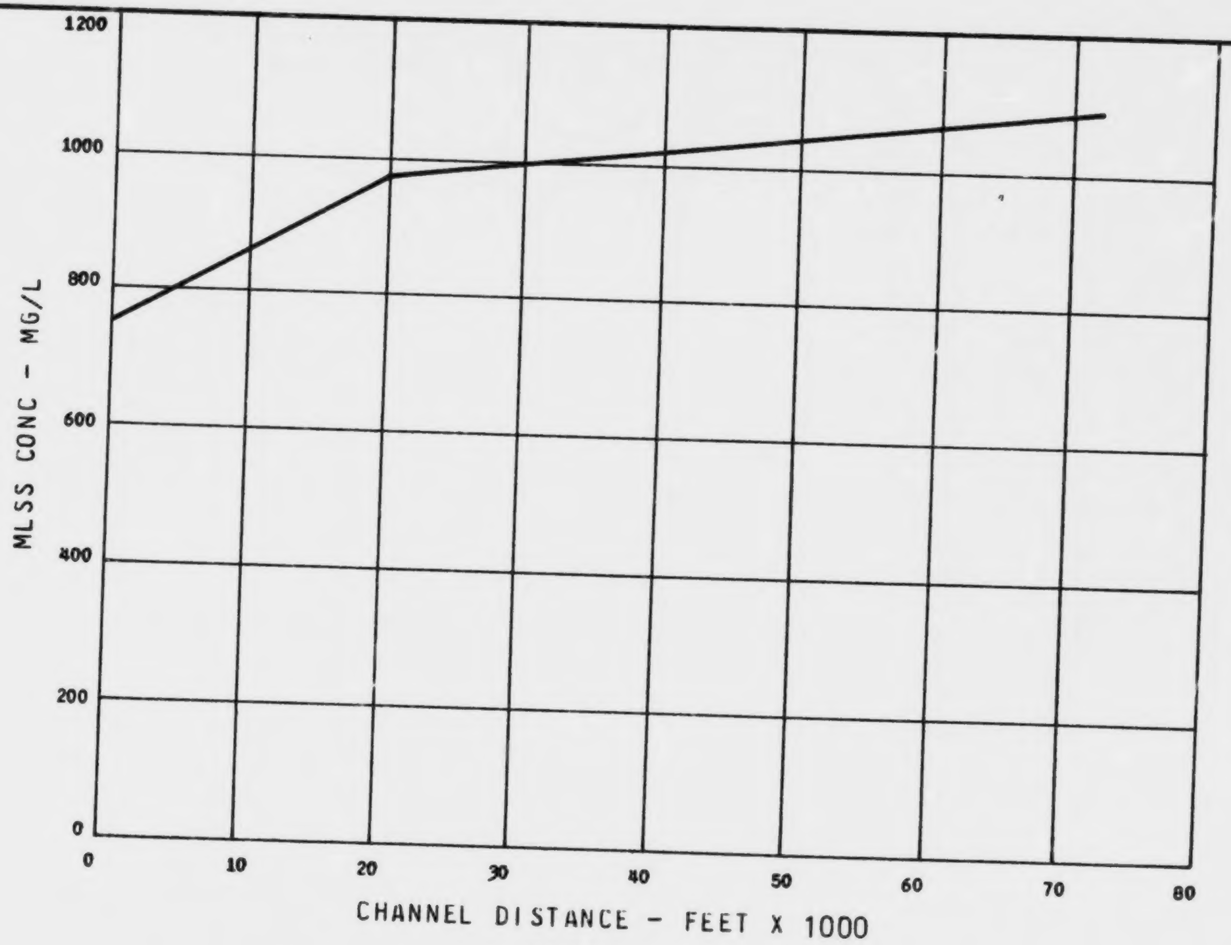


FIGURE 27
MLSS CONCENTRATION VS CHANNEL DISTANCE
MINIMUM LOAD - ALTERNATE 2

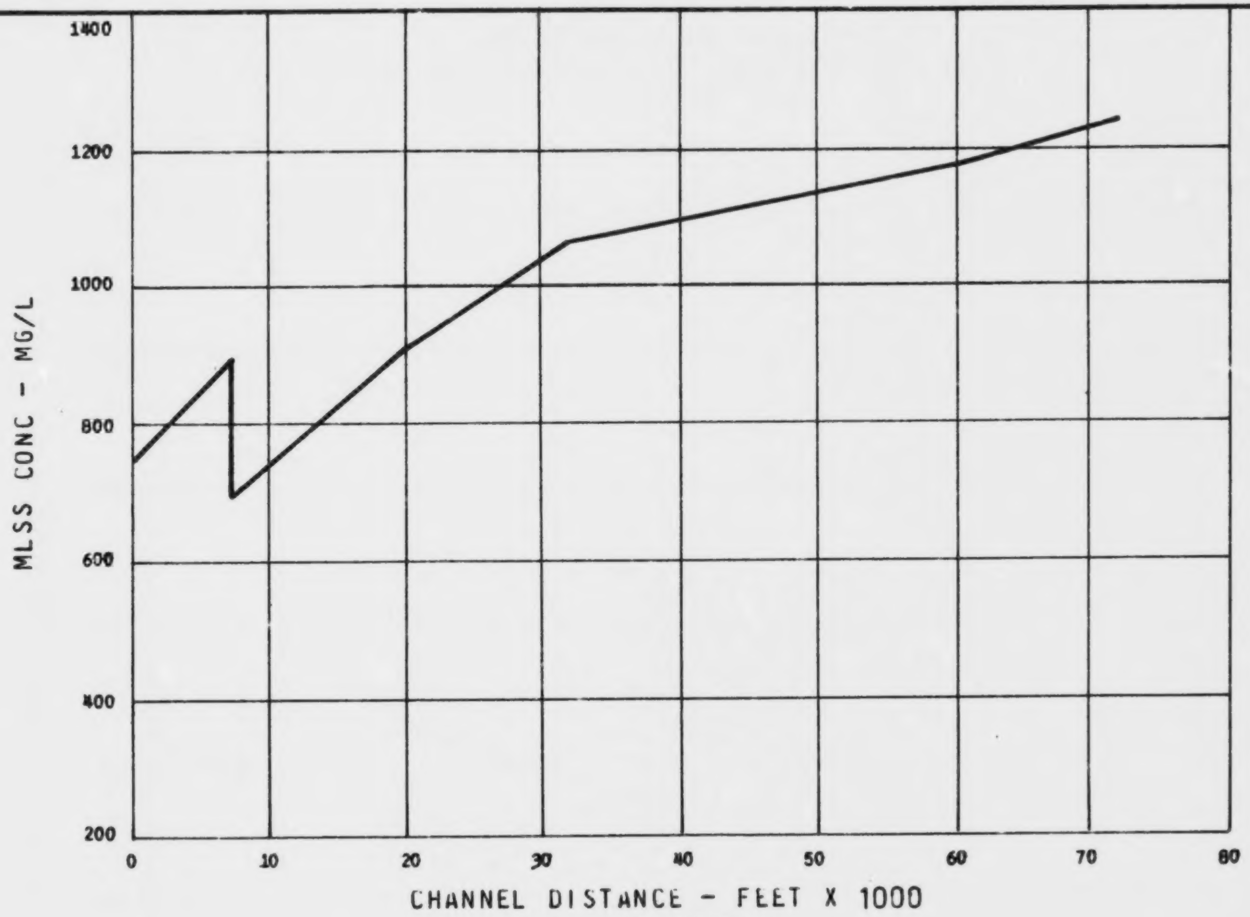


FIGURE 28
MLSS CONCENTRATION VS CHANNEL DISTANCE
MEDIUM LOAD - ALTERNATE 2

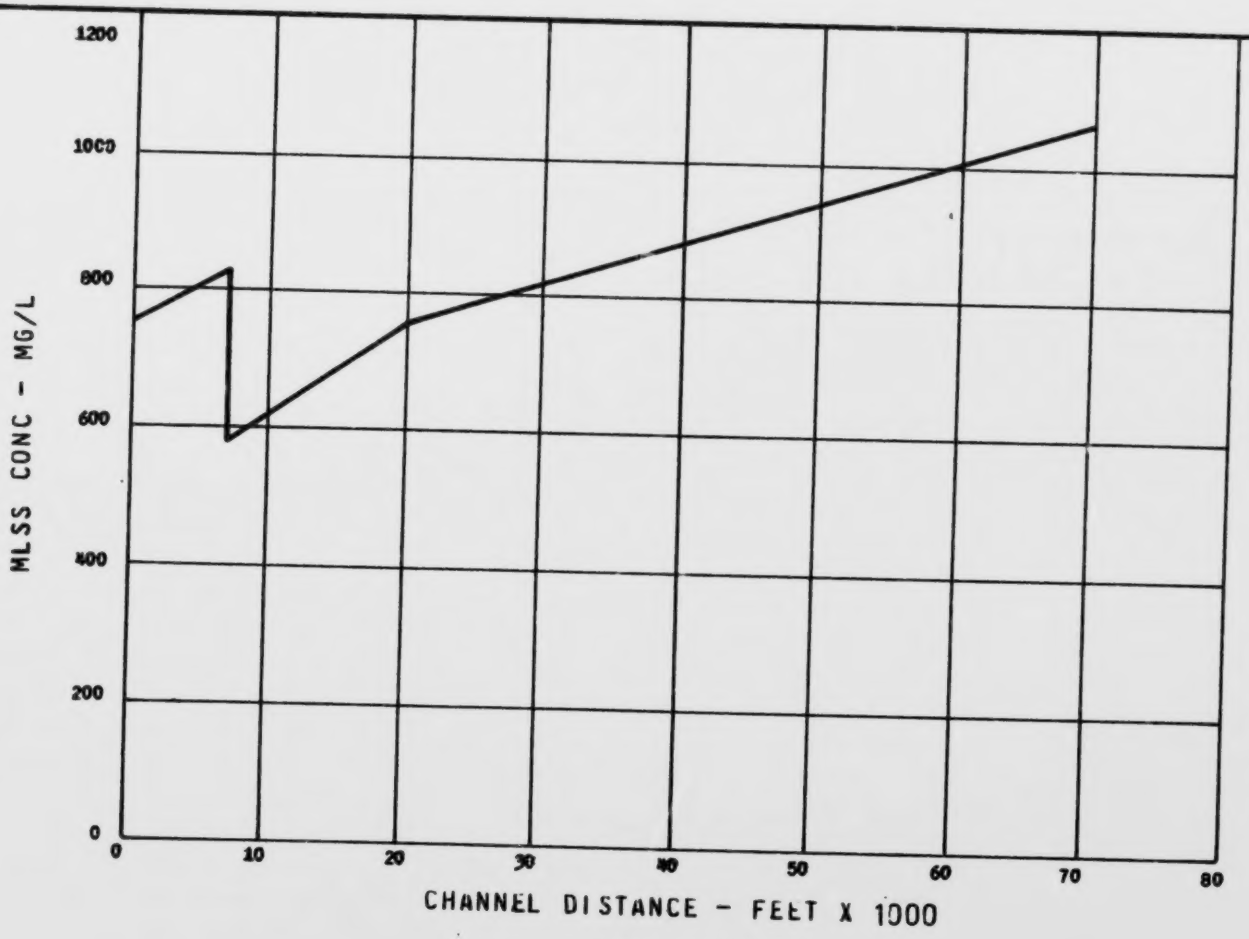


FIGURE 29
MLSS CONCENTRATION VS CHANNEL DISTANCE
MAXIMUM LOAD-ALTERNATE 2

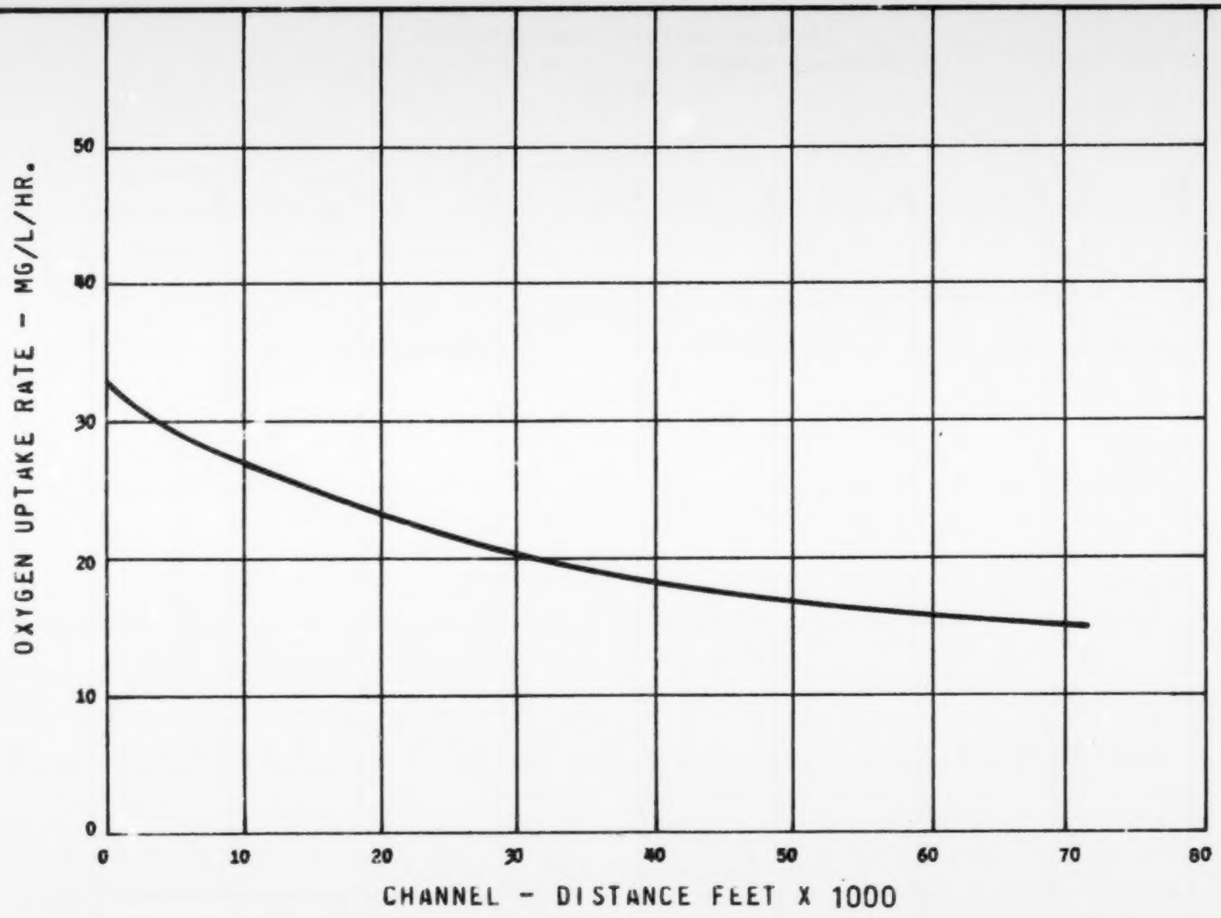


FIGURE 30
OXYGEN DEMAND VS CHANNEL DISTANCE
MINIMUM LOAD - ALTERNATE 2

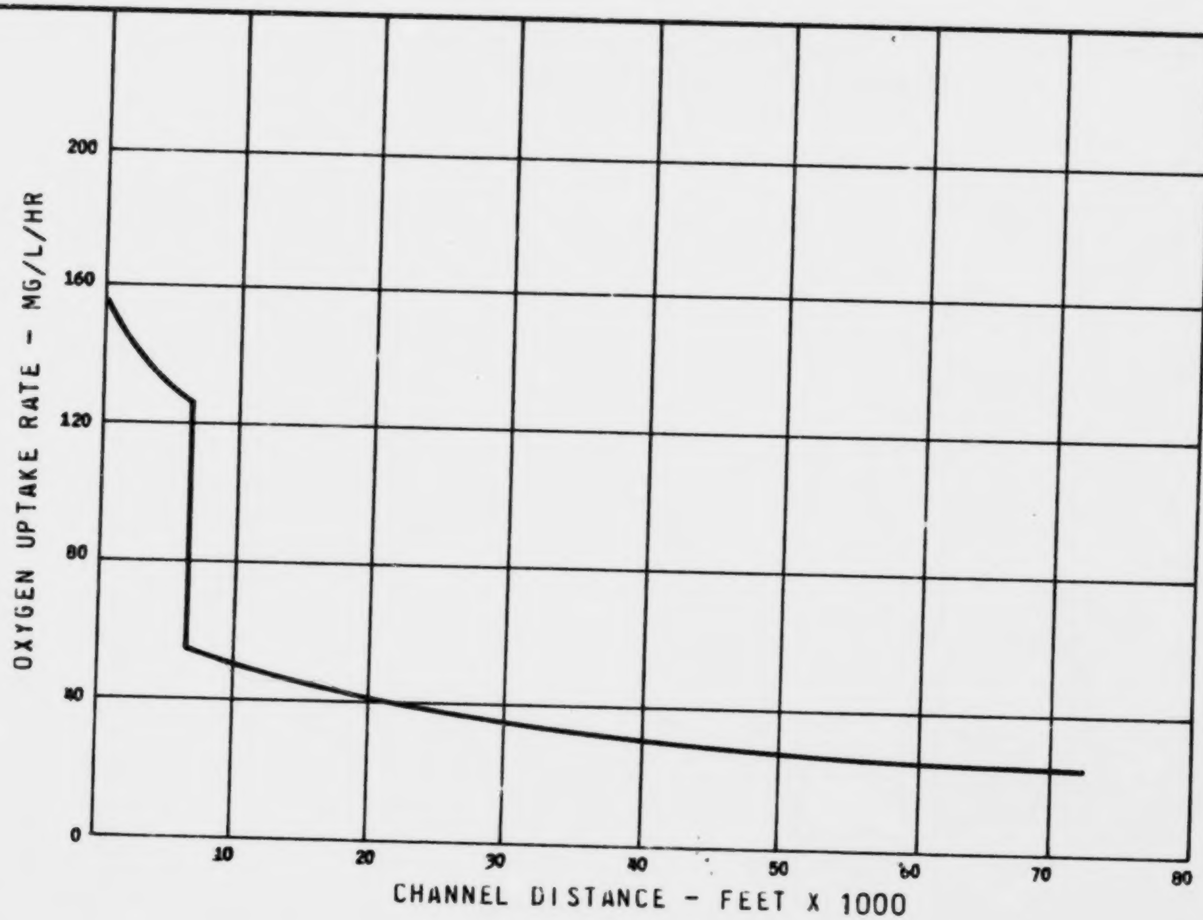


FIGURE 31
OXYGEN DEMAND VS CHANNEL DISTANCE
MEDIUM LOAD - ALTERNATE 2

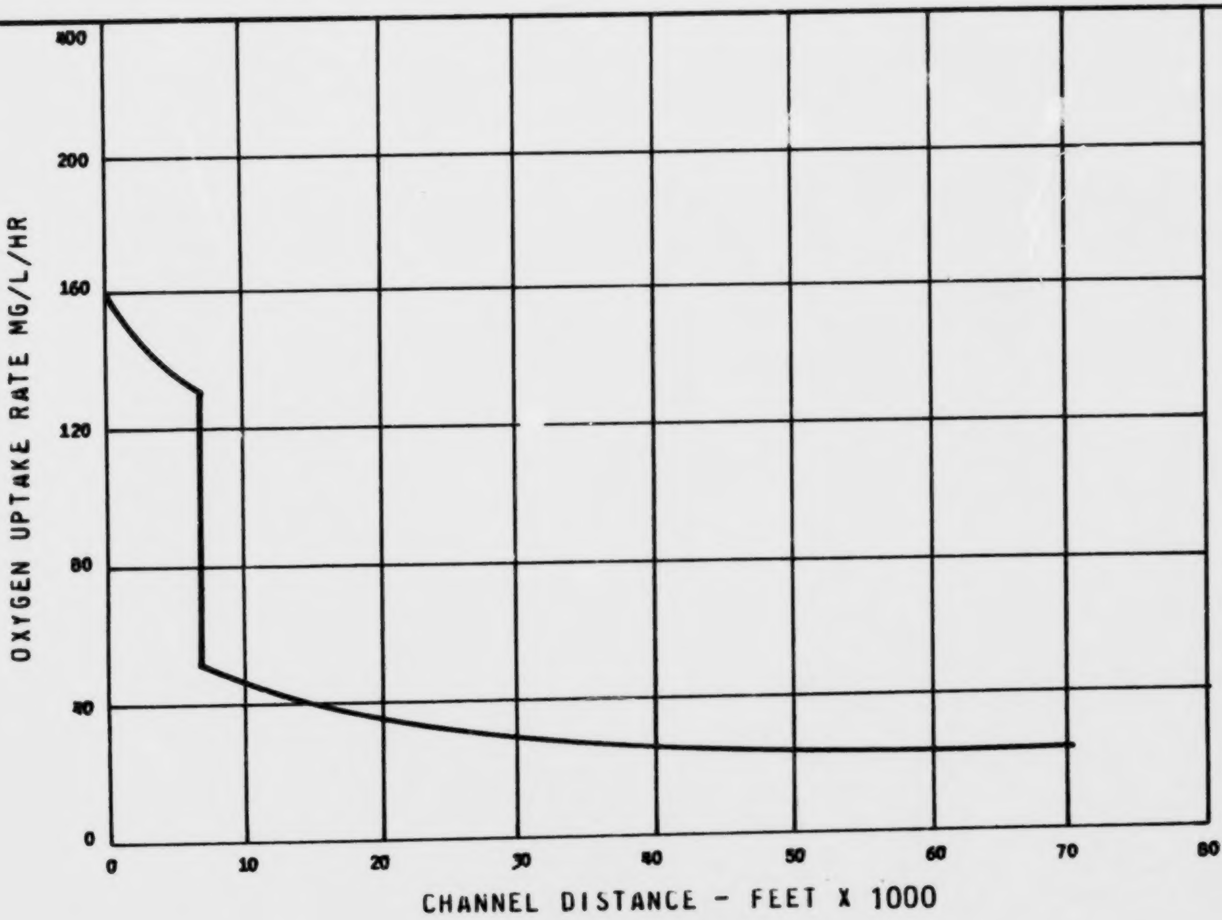


FIGURE 32
OXYGEN DEMAND VS CHANNEL DISTANCE
MAXIMUM LOAD ALTERNATE 2

TABLE 46

TOTAL AIR REQUIREMENT

MINIMUM LOAD

ALTERNATE 2

Reach	Blower Location	Ave Up Rt (r_r) mg/1/hr	Depth of Flow	D^*r_r $(C_L = 0.5)$	Diffuser Spacing-D	Channel Length-Ft	No of Diffusers	Blower Cap CFM
0-3500	3,500	33	2.7	1690	51	3,500	69	690
3500-7000	3,500	29	2.7	1690	58	3,500	61	610
7000-12000	12,000	27	2.7	1690	63	5,000	80	800
12000-16000	12,000	25	2.7	1690	68	4,000	59	590
16000-20000	20,000	24	2.7	1690	71	4,000	56	560
20000-24000	20,000	23	3.1	1550	67	4,000	60	600
24000-28000	28,000	22	3.1	1550	71	4,000	56	560
28000-32000	28,000	21	3.1	1550	74	4,000	54	540
32000-36000	36,000	20	3.1	1550	78	4,000	51	510
36000-40000	36,000	19	3.1	1550	82	4,000	49	490
40000-45000	45000	18	3.1	1550	86	5,000	58	580
45000-50000	45,000	17	3.1	1550	91	5,000	55	550
50000-55000	55,000	16	3.1	1550	97	5,000	52	520
55000-61000	55,000	16	3.1	1550	97	6,000	62	620
61000-66000	66,000	15	3.1	1550	103	6,000	58	580
66000-72000	66,000	15	3.1	1550	103	6,000	58	580

*Diffuser Spacing-Ft

TABLE 47

TOTAL AIR REQUIREMENTS

MEDIUM LOAD

ALTERNATE 2

Reach	Blower Location	Ave Up Rt (r_r) mg/l/hr	Depth of Flow	D^*r_r ($C_L = 0.5$)	Diffuser Spacing-D	Channel Length-Ft	No of Diffusers	Blower Cap CFM
0-3500	3,500	147	3.0	1580	11	3,500	318	3,180
3500-7000	3,500	132	3.0	1580	12	3,500	292	2,920
7000-12000	12,000	52	3.8	1340	26	5,000	192	1,920
12000-16000	12,000	47	3.8	1340	29	4,000	138	1,380
16000-20000	20,000	43	3.8	1340	31	4,000	129	1,290
20000-24000	20,000	40	4.4	1210	30	4,000	133	1,330
24000-28000	28,000	37	4.4	1210	33	4,000	121	1,210
28000-32000	28,000	35	4.4	1210	35	4,000	114	1,140
32000-36000	36,000	33	4.4	1210	37	4,000	108	1,080
36000-40000	36,000	31	4.4	1210	39	4,000	102	1,020
40000-45000	45,000	30	4.4	1210	40	5,000	125	1,250
45000-50000	45,000	29	4.4	1210	42	5,000	119	1,190
50000-55000	55,000	28	4.4	1210	43	5,000	116	1,160
55000-61000	55,000	27	4.4	1210	45	6,000	133	1,330
61000-66000	66,000	26	4.4	1210	46	6,000	130	1,300
66000-72000	66,000	26	4.4	1210	47	6,000	128	1,280

*Diffuser Spacing in Ft

TABLE 48

TOTAL AIR REQUIREMENTS

MAXIMUM LOAD

ALTERNATE 2

Reach	Blower Location	Ave Up Rt (r_r) mg/l/hr	Depth of Flow	D^*r_r ($C_L = 0.5$)	Diffuser Spacing-D	Channel Length-Ft	No of Diffusers	Blower Cap CFM
0-3500	3,500	152	3.2	1510	10	3,500	350	3,500
3500-7000	3,500	135	3.2	1510	12	3,500	290	2,900
7000-12000	12,000	48	4.8	1150	24	5,000	208	2,080
12000-16000	12,000	44	4.8	1150	26	4,000	154	1,540
16000-20000	20,000	40	4.8	1150	29	4,000	138	1,380
20000-24000	20,000	37	5.6	1040	28	4,000	143	1,430
24000-28000	28,000	34	5.6	1040	31	4,000	129	1,290
28000-32000	28,000	32	5.6	1040	33	4,000	121	1,210
32000-36000	36,000	30	5.6	1040	35	4,000	114	1,140
36000-40000	36,000	28	5.6	1040	37	4,000	108	1,080
40000-45000	45,000	26	5.6	1040	40	5,000	125	1,250
45000-50000	45,000	25	5.6	1040	42	5,000	119	1,190
50000-55000	55000	24	5.6	1040	43	5,000	116	1,160
55000-61000	55,000	24	5.6	1040	43	6,000	139	1,390
61000-66000	66,000	24	5.6	1040	44	6,000	136	1,360
66000-72000	66,000	24	5.6	1040	44	6,000	136	1,360

*Diffuser Spacing in Ft

TABLE 49

BLOWER AND AIR LINE SIZING

MINIMUM LOAD

ALTERNATE 2

Blower Location	Blower Capacity	Discharge PS	Blower HP	Air Line Length	Line Size	Length Served
3,500	690	9	37	3,500	6"	0-3500
3,500	610	8	37	3,500	5"	3500-7000
12,000	800	8	37	5,000	6"	7000-12000
12,000	590	8	37	4,000	5"	12000-16000
20,000	560	8	34	4,000	5"	16000-20000
20,000	600	8	37	4,000	5"	20000-24000
28,000	560	8	34	4,000	5"	24000-28000
28,000	540	8	34	4,000	5"	28000-32000
36,000	510	8	34	4,000	5"	32000-36000
36,000	490	8	34	4,000	5"	36000-40000
45,000	580	8	34	5,000	5"	40000-45000
45,000	550	8	34	5,000	5"	45000-50000
55,000	520	8	34	5,000	5"	50000-55000
55,000	620	8	37	5,000	5"	55000-60000
66,000	580	8	34	6,000	5"	60000-66000
66,000	580	8	34	6,000	5"	66000-72000
			562 HP			

TABLE 50

BLOWER AND AIR LINE SIZING

MEDIUM LOAD

ALTERNATE 2

Blower Location	Blower Capacity	Discharge PS	Blower HP	Air Line Length	Line Size	Length Served
3,500	3180	8	154	3,500	10"	0-3500
3,500	2920	8	141	2,500	10'	3500-7000
12,000	1920	8	93	5,000	10"	7000-12000
12,000	1380	8	64	4,000	8"	12000-16000
20,000	1290	8	60	4,000	8"	16000-20000
20,000	1330	8	60	4,000	8"	20000-24000
28,000	1210	8	60	4,000	8"	24000-28000
28,000	1140	8	60	4,000	8"	28000-32000
36,000	1080	8	60	4,000	8"	32000-36000
36,000	1020	8	46	4,000	8"	36000-40000
45,000	1250	8	60	5,000	8"	40000-45000
45,000	1190	8	60	5,000	8"	45000-50000
55,000	1160	8	60	5,000	8"	50000-55000
55,000	1330	8	60	5,000	8"	55000-60000
66,000	1300	8	60	6,000	8"	60000-66000
66,000	1280	8	60	6,000	8"	66000-72000
			1,158 HP			

TABLE 51

BLOWER AND AIR LINE SIZING

MAXIMUM LOAD

ALTERNATE 2

Blower Location	Blower Capacity	Discharge PS	Blower HP	Air Line Length	Line Size	Length Served
3,500	3,500	8	154	3,500	10"	0-3500
3,500	2,900	8	141	3,500	10"	3500-7000
12,000	2,080	8	117	5,000	10"	7000-12000
12,000	1,540	8	70	4,000	8"	12000-16000
20,000	1,380	8	64	4,000	8"	16000-20000
20,000	1,430	8	64	4,000	8"	20000-24000
28,000	1,290	8	60	4,000	8"	24000-28000
28,000	1,210	8	60	4,000	8"	28000-32000
36,000	1,140	8	60	4,000	8"	32000-36000
36,000	1,080	8	60	4,000	8"	36000-40000
45,000	1,250	8	60	5,000	8"	40000-45000
45,000	1,190	8	60	5,000	8"	45000-50000
55,000	1,160	8	60	5,000	8"	50000-55000
55,000	1,390	8	64	5,000	8"	55000-60000
66,000	1,360	8	64	6,000	8"	60000-66000
66,000	1,360	8	64	6,000	8"	66000-72000
			1,222 HP			

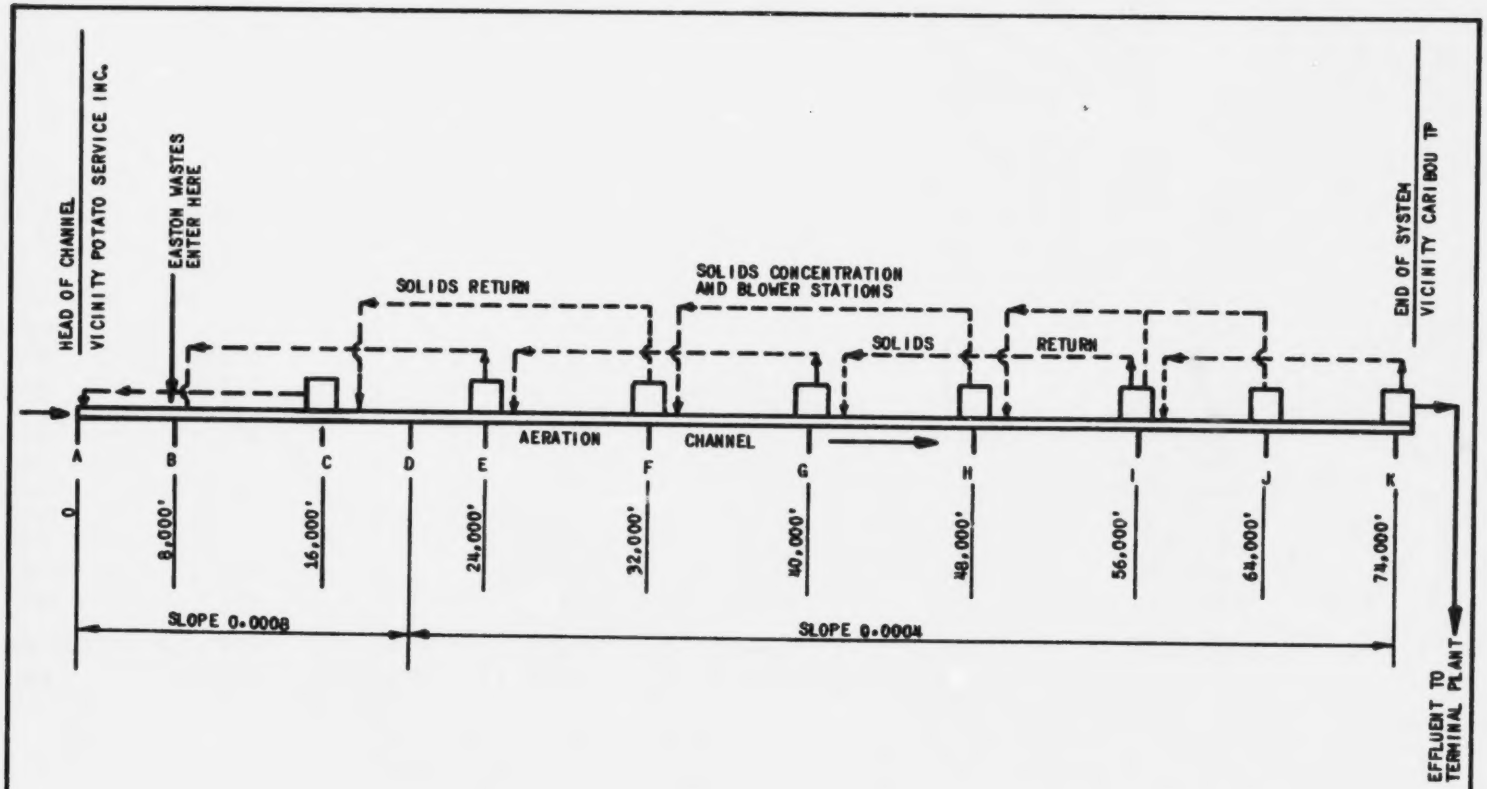
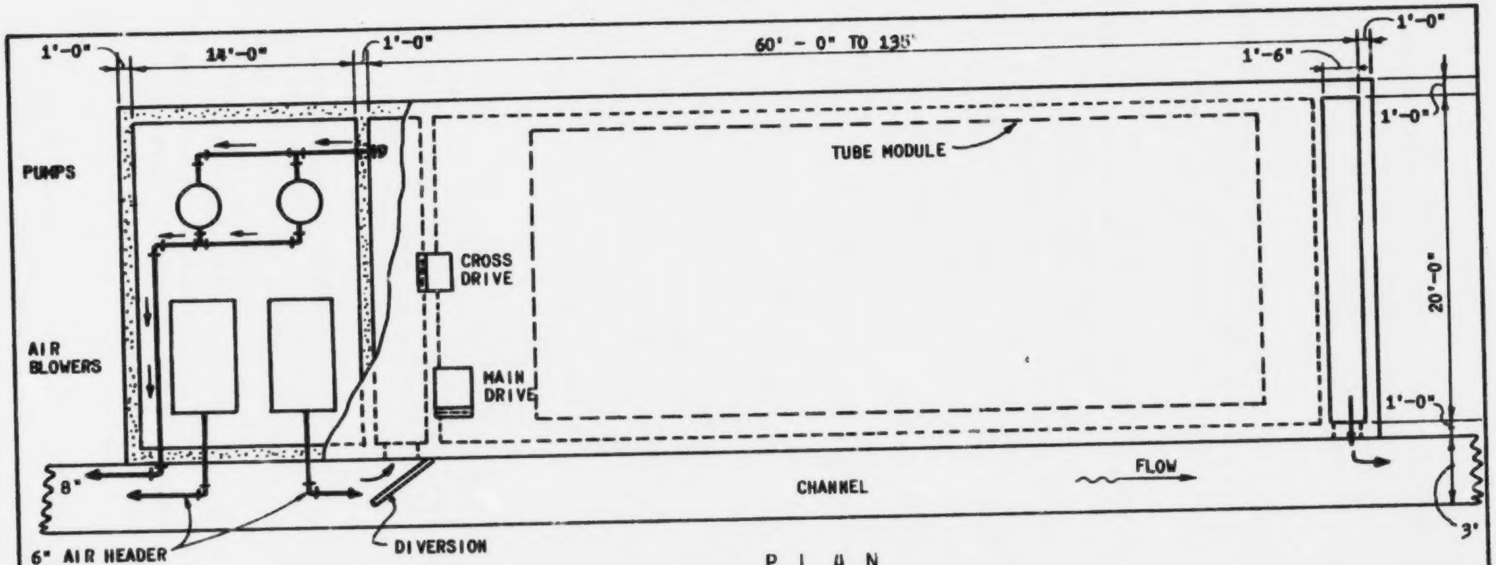

HEAD OF CHANNEL
VICINITY POTATO SERVICE INC.
EASTON WASTES ENTER HERE
SOLIDS RETURN
SOLIDS CONCENTRATION AND BLOWER STATIONS
SOLIDS
RETURN
END OF SYSTEM
VICINITY CARIBOU TP
AERATION
CHANNEL
A
B
C
D
E
F
G
H
I
J
K
0
8,000'
16,000'
24,000'
32,000'
40,000'
48,000'
56,000'
64,000'
74,000'
SLOPE 0.0008
SLOPE 0.0004
EFFLUENT TO TERMINAL PLANT

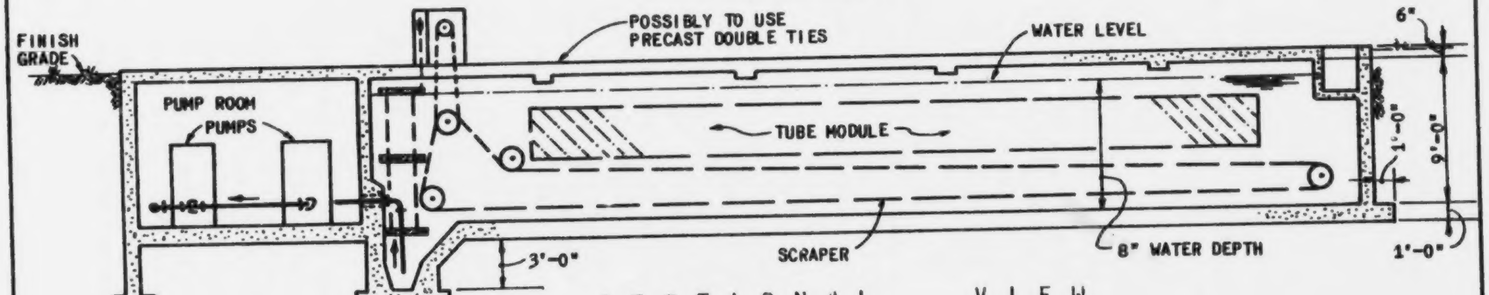
FIGURE 33

SCHEMATIC DIAGRAM

TREATMENT-TRANSPORT SYSTEMS-ALTERNATE 3


1'-0"
14'-0"
1'-0"
60' - 0" TO 135'
1'-6"
1'-0"
PUMPS
AIR BLOWERS
CROSS DRIVE
MAIN DRIVE
TUBE MODULE
1'-0"
20'-0"
1'-0"
8"
CHANNEL
FLOW
3'
6" AIR HEADER
DIVERSION

P L A N


FINISH GRADE
PUMP ROOM
PUMPS
POSSIBLY TO USE PRECAST DOUBLE TIES
WATER LEVEL
6"
TUBE MODULE
1'-0"
9'-0"
3'-0"
SCRAPER
8" WATER DEPTH
1'-0"

S E C T I O N A L   V I E W

FIGURE 34
TYPICAL SOLIDS CONCENTRATOR AND RETURN INSTALLATION
TREATMENT-TRANSPORT SYSTEMS

As previously discussed under "System Configuration", the oxygen demand rate of the recirculated sludge will be in the order of 40 mg/l/hr. The design of the sludge recirculation system is based on the assumption that sludge activity will not be seriously affected without aeration for 1.5 to 2 hours.

The basic design for the above biological system in terms of BOD removals and solids growth, was based on equations (7) and (8), as previously presented for the Alternate 2 biological system, and on the design criteria above.

The demand for oxygen will vary along the channel with the higher demands at the head end and declining values as the flow progresses down channel. The variation of demand along an extended channel of this nature cannot be predicted precisely without additional pilot plant or laboratory studies specifically designed for the system. For preliminary analysis the data suggested by McCabe and Eckenfelder[41] on demand variation with time was used. This data, as adapted to the channel section, is shown on Figure 35. It is emphasised that this data needs confirmation by additional study prior to final system design. Based on this data and other design criteria discussed previously, the variation in oxygen demand along the channel has been computed for each loading condition.

Tables 52, 53, and 54 indicate the volume, length and depth relationships for each reach of the channel for the three design load conditions. In each case a design MLSS concentration of 3000 mg/l was chosen for preliminary design. Plots of BOD/MLSS ratio versus MLSS concentrations are shown on Figures 36, 37 and 38. Based on this data and the design criteria listed previously, the estimated BOD removals and solids production in the channel were computed by the methods previously presented. Tables 55, 56, and 57 summarize the biologic characteristics of the channel. Figures 39, 40 and 41 show the estimated mixed liquor suspended solids variations along the channel.

Review of Tables 55, 56 and 57 reveal that BOD reductions of over 90% may be anticipated in the channel section for each of the design loading conditions. Solids production will vary with the total BOD removal and ranges from about 10,000 pounds per day at minimum loading to nearly 50,000 pounds per day at maximum loading conditions.

Accomplishment of the BOD removals suggested is dependent upon adequate oxygen transfer to maintain aerobic biologic activity. The introduction of oxygen into the mixed liquor in the channel will be accomplished via a combination of surface transfer and diffused air placed in the channel. Analysis indicates that without artificial aeration, the mixed liquor would quickly become devoid of oxygen indicating surface aeration alone is not sufficient. This would be expected in the relatively narrow channel section. Surface aeration could be increased by widening the section, but velocity problems prohibit this.

The oxygen demand analyses, as shown in Figures 42, 43, and 44, suggest that peak oxygen demands could range up to about 70 mg/l/hr with maximum design loading, dropping to a low of about 15 mg/l/hr at the end of the channel with minimum loading conditions.

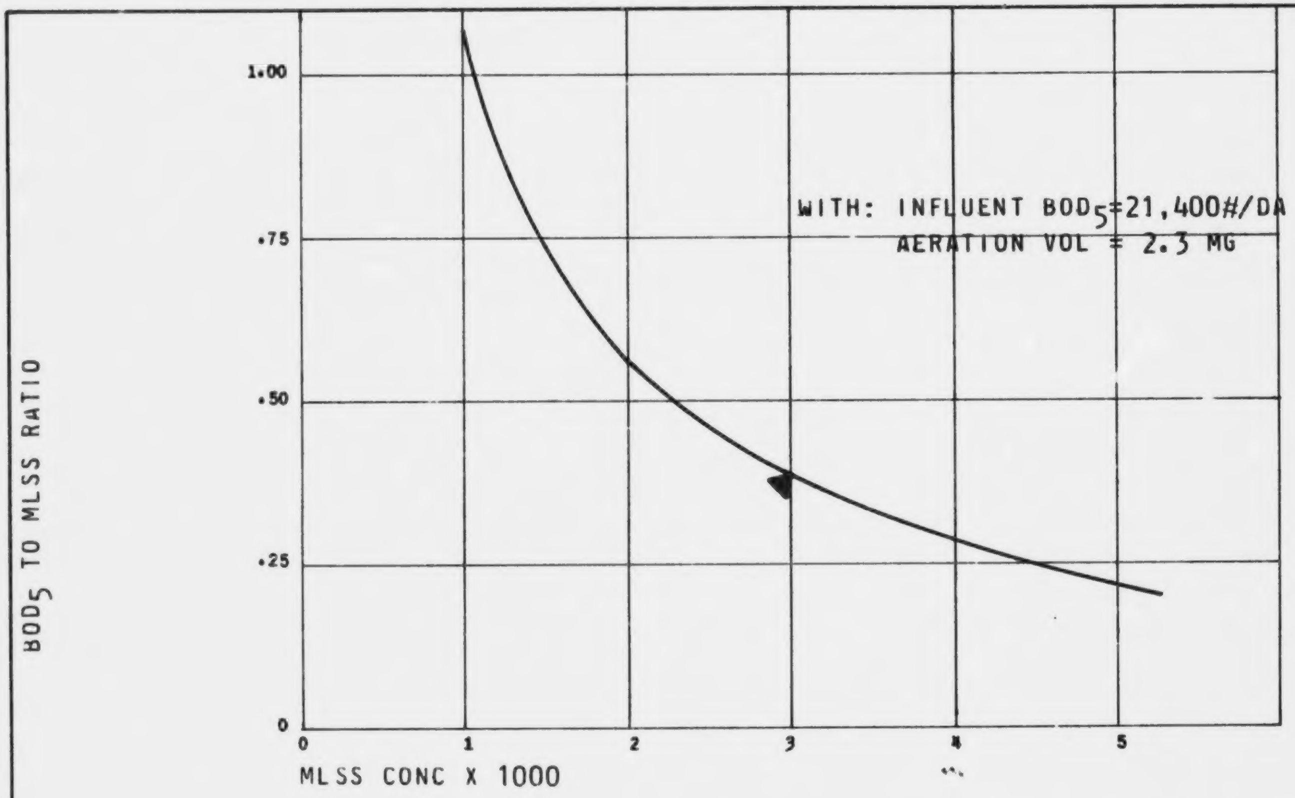


FIGURE 35
BOD/MLSS RATIO VS MLSS CONCENTRATION
MINIMUM CONDITIONS

TABLE 52

CHANNEL HYDRAULIC CHARACTERISTICS

MINIMUM LOADING

ALTERNATE 3

Inflow Rate = 5.26 CFS
Sol Ret Rate = 1.32 CFS

Channel Reach	Slope ft/ft	Depth of Flow-ft	Vol Per Ft(CF)	Vel ft/sec	Length ft	Reach Vol(CF)	Reach Det Time(hrs)
A to B	0.0008	2.9	2.9	2.20	8,000	23,200	1.01
B to C	0.0008	3.2	3.3	2.26	8,000	26,400	0.98
C to D	0.0008	3.2	3.3	2.26	4,000	13,200	0.49
D to E	0.0004	4.0	4.7	1.68	4,000	18,800	0.66
C to E					8,000	32,000	1.15
E to F	0.0004	4.0	4.7	1.68	8,000	37,600	1.32
F to G	0.0004	4.0	4.7	1.68	8,000	37,600	1.32
G to H	0.0004	4.0	4.7	1.68	8,000	37,600	1.32
H to I	0.0004	4.0	4.7	1.68	8,000	37,600	1.32
I to J	0.0004	4.0	4.7	1.68	8,000	37,600	1.32
J to K	0.0004	3.5	3.9	1.64	10,000	39,000	1.70

TABLE 53

CHANNEL HYDRAULIC CHARACTERISTICS

MEDIUM LOADING

ALTERNATE 3

Ave Inflow Rate = 9.6 CFS
Return Rate = 2.4 CFS

Channel Reach	Slope	Depth of Flow-ft	Vol Per Ft(CF)	Vel ft/sec	Reach Length-ft	Reach CF	Reach Det Time-hrs
A to B	0.0008	3.5	3.9	2.30	8,000	31,200	0.97
B to C	0.0008	4.7	6.1	2.46	8,000	48,800	0.90
C to D	0.0008	4.7	6.1	2.46	4,000	24,400	0.45
D to E	0.0004	5.6	8.0	1.86	4,000	43,000	0.60
C to E					8,000	56,400	1.05
E to J	0.0004	5.6	8.0	1.86	40,000	320,000	6.00
J to K	0.0004	5.0	6.7	1.80	10,000	67,000	1.55

CHANNEL HYDRAULIC CHARACTERISTICS

MAXIMUM LOADING

ALTERNATE 3

Inflow Rate = 14.9 CFS
Return Rate = 3.7 CFS

Channel Reach	Slope	Depth of Flow-Ft	Vol Per Ft-CF	Vel ft/sec	Reach Length-ft	Reach CF	Reach Det Time-hrs
A to B	0.0008	3.7	4.2	2.32	8,000	33,600	0.96
B to C	0.0008	6.0	8.8	2.64	8,000	70,500	0.34
C to D	0.0008	6.0	8.8	2.64	4,000	35,200	0.42
D to E	0.0004	6.9	10.8	2.04	4,000	43,200	0.54
C to E							0.96
E to J	0.0004	6.9	10.8	2.04	40,000	432,000	5.45
J to K	0.0004	6.1	9.0	1.94	10,000	90,000	1.43
						704,500	9.64

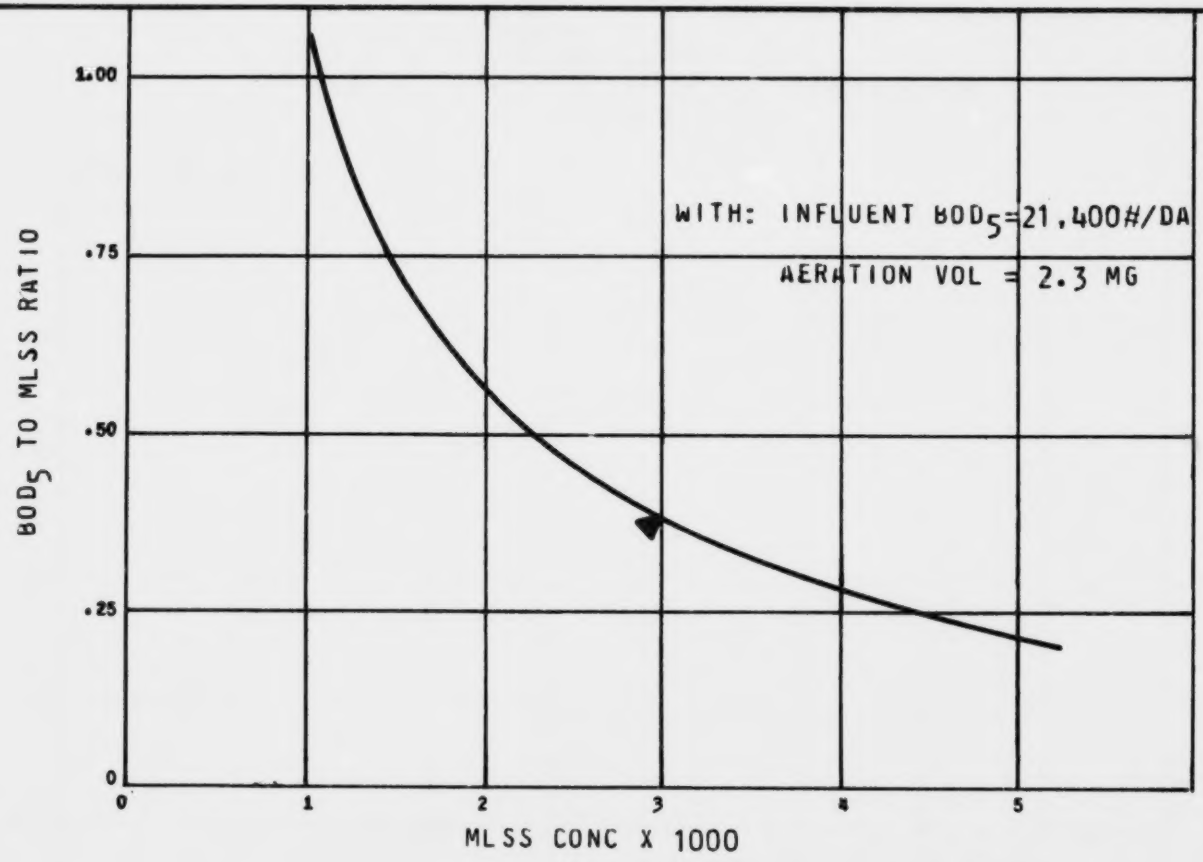


FIGURE 36
BOD MLSS RATIO VS MLSS CONCENTRATION
MINIMUM LOAD
ALTERNATE 3

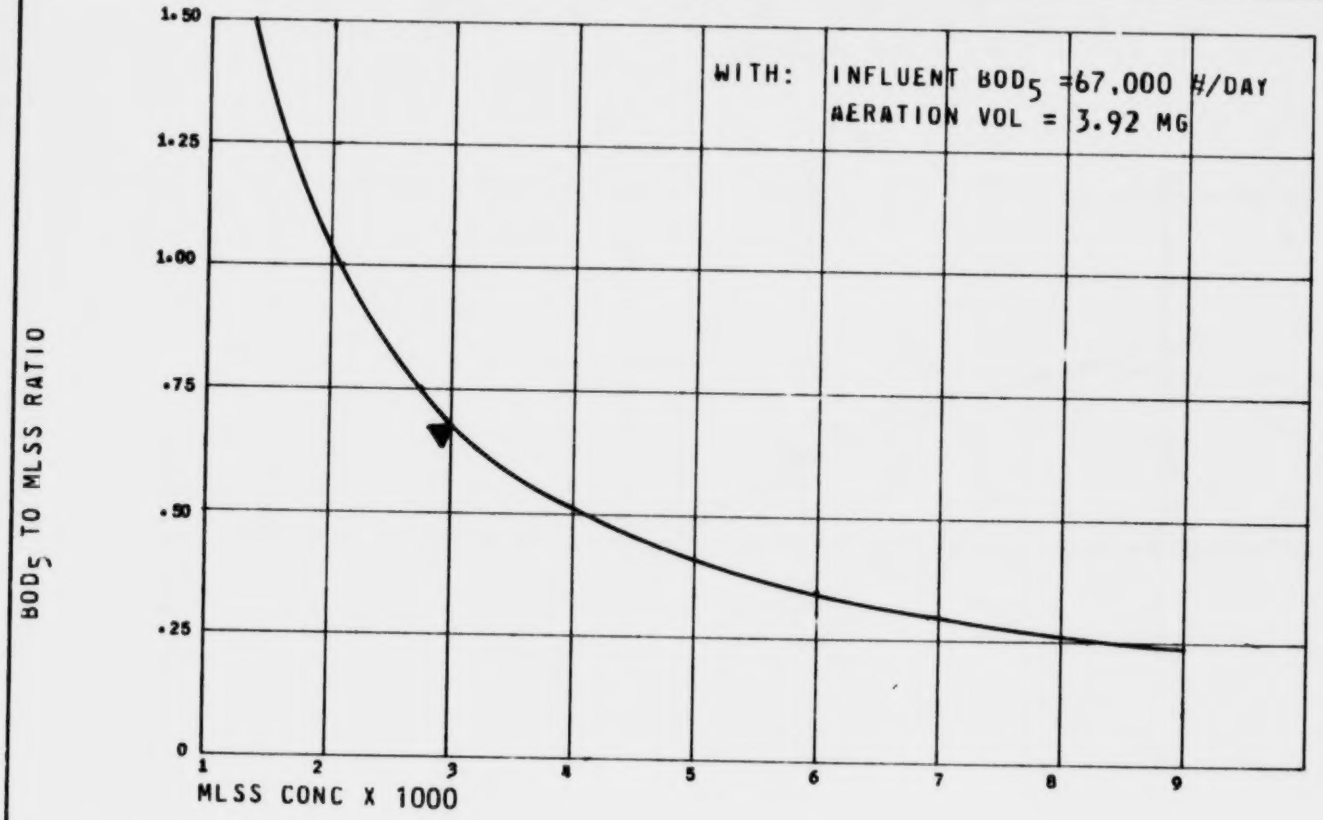


FIGURE 37
BOD/MLSS RATIO VS MLSS CONCENTRATION
MEDIUM LOAD
ALTERNATE 3

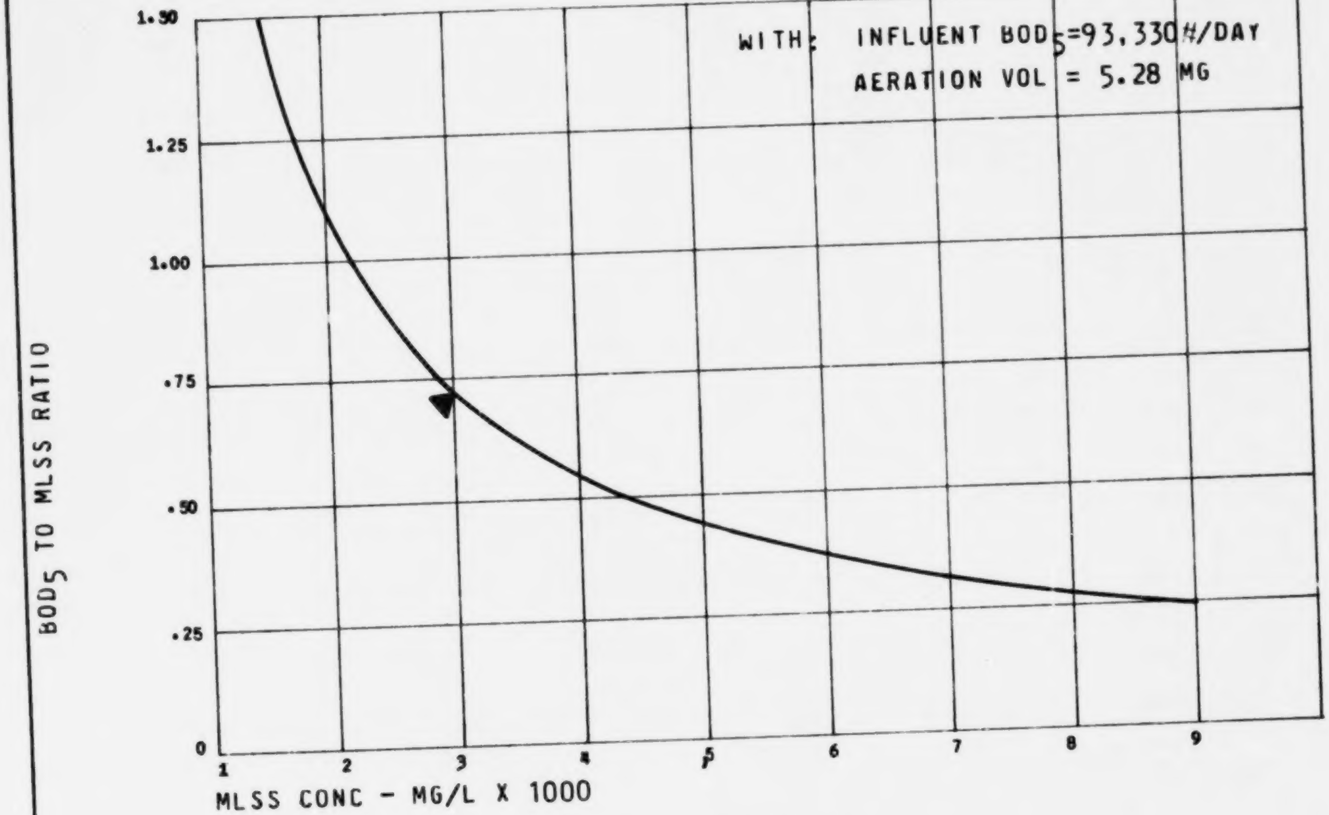


FIGURE 38
BOD/MLSS RATIO VS MLSS CONCENTRATION
MAXIMUM LOAD
ALTERNATE 3

TABLE 55

CHANNEL BIOLOGIC CHARACTERISTICS

MINIMUM LOAD

ALTERNATE 3

Channel Reach	Det Time hrs	Reach Vol MG	MLSS Ave Reach Conc(mg/l)	MLSS Tot Lbs In System	BOD_5 Removal %	BOD_5 Removal lbs	BOD_5 Start mg/l	BOD_5 Start lbs	BOD_5 End mg/l	BOD_5 End lbs	Sludge Production lbs/day
A to B	1.0	.174	1,800	60,000	43	9,200	750	21,400	410	12,200	3,000
B to C	1.0	.198	3,000	60,000			410	12,200			
A to C	2.0	.372	2,450	60,000	66	13,400	750	21,400	280	8,000	5,700
C to E	1.2	.240	3,000	60,000							
A to E	3.2	.612	2,700	60,000	77	16,400	750	21,400	175	5,000	7,700
E to J	6.6	1.41	3,000								
A to J	9.8	2.02	2,900	60,000	91	19,500	750	21,400	67	1,900	9,700
J to K	1.7	0.29	2,100	60,000							
A to K	11.5	2.31	2,800	60,000	92	19,700	750	21,400	60	1,700	9,800

ACTUAL MLSS CONC

PT		mg/l
A	=	1,800
B_1	=	1,885
B_2	=	3,085
C	=	3,160
E	=	3,220
J_1	=	3,280
J_2	=	2,080
K	=	2,100

TABLE 56

CHANNEL BIOLOGIC CHARACTERISTICS

MEDIUM LOAD

ALTERNATE 3

Channel Reach	Det Time hrs	Reach Vol MG	MLSS Ave Reach Conc (mg/1)	MLSS Tot lbs To System	BOD_5 Removal %	BOD_5 Removal lbs	BOD_5 Start mg/1	BOD_5 Start lbs	BOD_5 End mg/1	BOD_5 End lbs	Sludge Production lbs/day
A to B	0.97	0.234	2,000	110,000	43	19,000[2]	1,300	45,600	750	26,000	7,000
B to C	0.90	0.366	3,000	110,000							
A to C	1.87(1.56)[2]	0.600	2,600	110,000	62	41,500	1,300	67,000	500	25,500	21,500
C to E	1.05	0.423	3,000								
A to E	2.92(2.62)[2]	1.023	2,750	110,000	74	51,000	1,300	67,000	310	16,000	27,700
E to J	6.00	2.400	3,000	110,000							
A to J	8.92(8.62)[2]	3.42	2,900	110,000	91	61,000	1,300	67,000	115	6,000	34,000
J to K	1.55	0.50	1,500	110,000							
A to K	10.47(10.20)[2]	3.92	2,700	110,000	92	61,500	1,300	67,000	105	5,500	34,500

ACTUAL MLSS CONC

A = 2,000
B_1 = 2.145
B_2 = 3,090
C = 3,270
E = 3,360
J_1 = 3,440
J_2 = 2,030
K = 2,050

---

[1] Based on BOD input at P I
[2] Weighted Ave based on BOD Loads

TABLE 57

CHANNEL BIOLOGIC CHARACTERISTICS

MAXIMUM LOAD

ALTERNATE 3

Channel Reach	Det Time hrs	Reach Vol MG	MLSS Ave Reach Conc (mg/1)	MLSS Tot Lbs To System	BOD_5 Removal %	BOD_5 Removal Lbs	BOD_5 Start mg/1	BOD_5 Start lbs	BOD_5 End mg/1	BOD_5 End lbs	Sludge Production lbs/day
A to B	0.96	0.252	2,760	130,000	51	27,000[1]	1,380	51,900		24,900	11,000
B to C	0.84	0.528	3,000	130,000							
A to C	1.80(1.38)[2]	0.780	2,920	130,000	61.5	57,500	1,170	93,300		35,800	31,000
C to E	0.96	0.588	3,000								
A to E	2.76(2.39)[2]	1.368	2,930	130,000	74	69,000	1,170	93,000		24,300	38,500
E to J	5.45	3.240	3,000								
A to J	8.21(7.85)[2]	4.608	2,990	130,000	90.5	84,500	1,170	93,000		8,800	48,500
J to K	1.43	0.675	2,020								
A to K	9.64(9.25)	5.283	2,880	130,000	91.5	85,500	1,170	93,300	93	7,800	49,000

ACTUAL MLSS CONC

A = 2,760
B_1 = 3,050
B_2 = 3,170
C = 3,260
E = 3,320
J_1 = 3,400
J_2 = 2,500
K = 2,510

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[1] Based on BOD input at P.I.
[2] Weighted Ave. Based on BOD loads

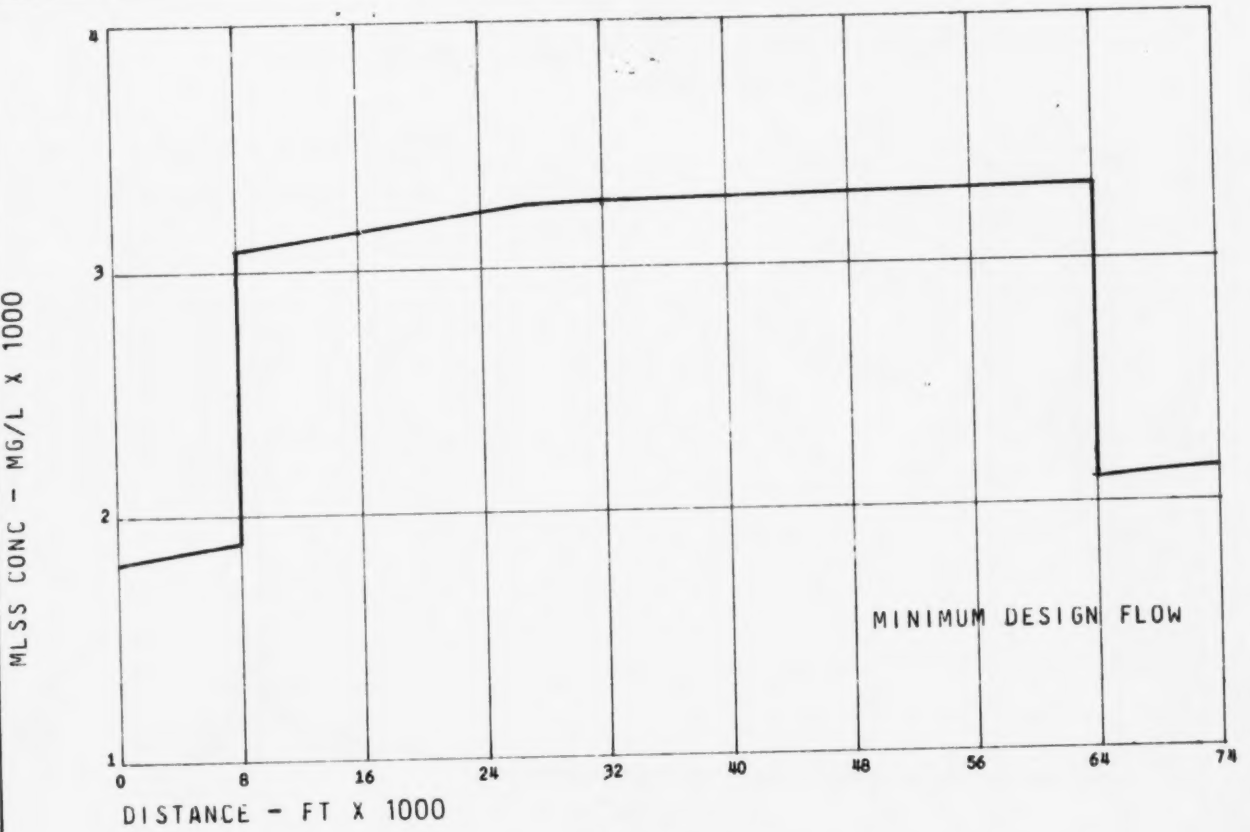


FIGURE 39
MLSS CONCENTRATION VS DISTANCE
ALTERNATE 3

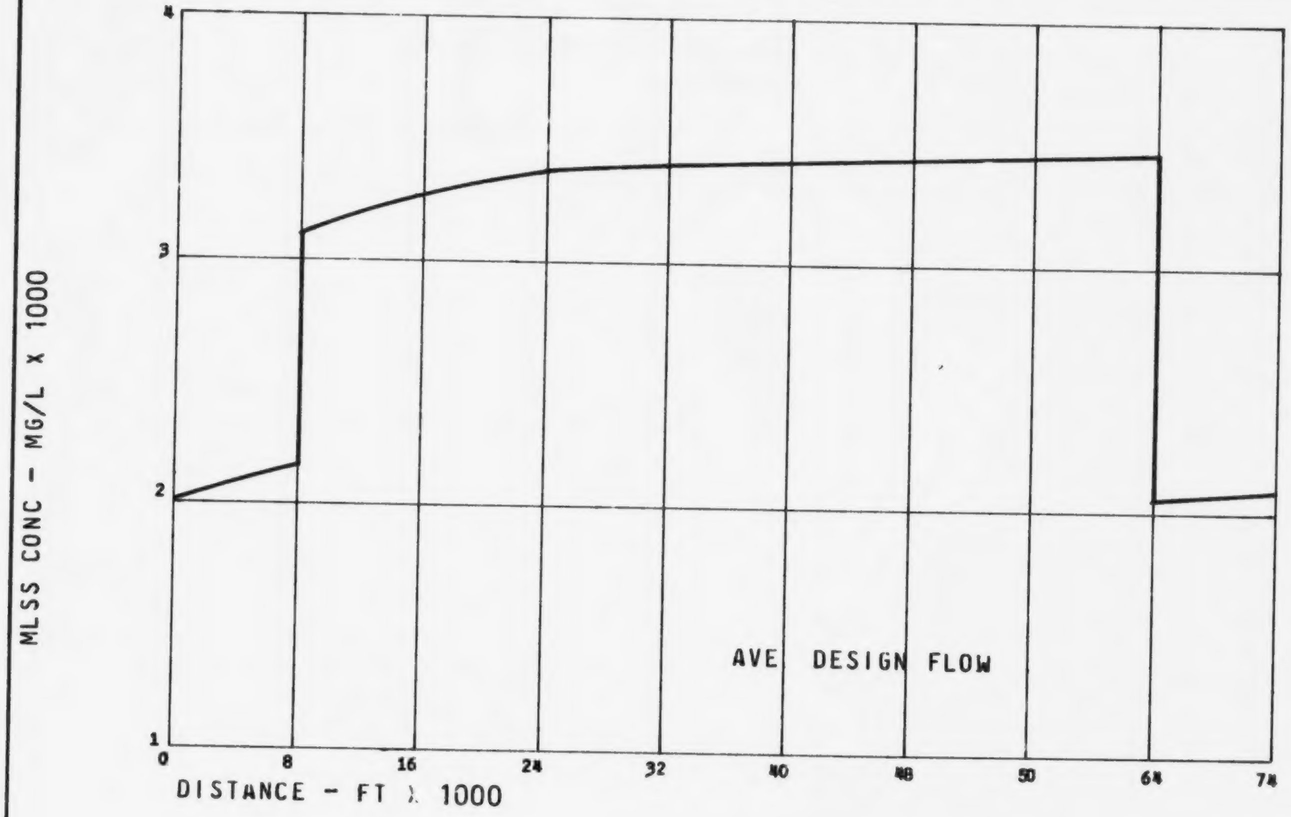


FIGURE 40
MLSS CONCENTRATION VS DISTANCE
ALTERNATE 3

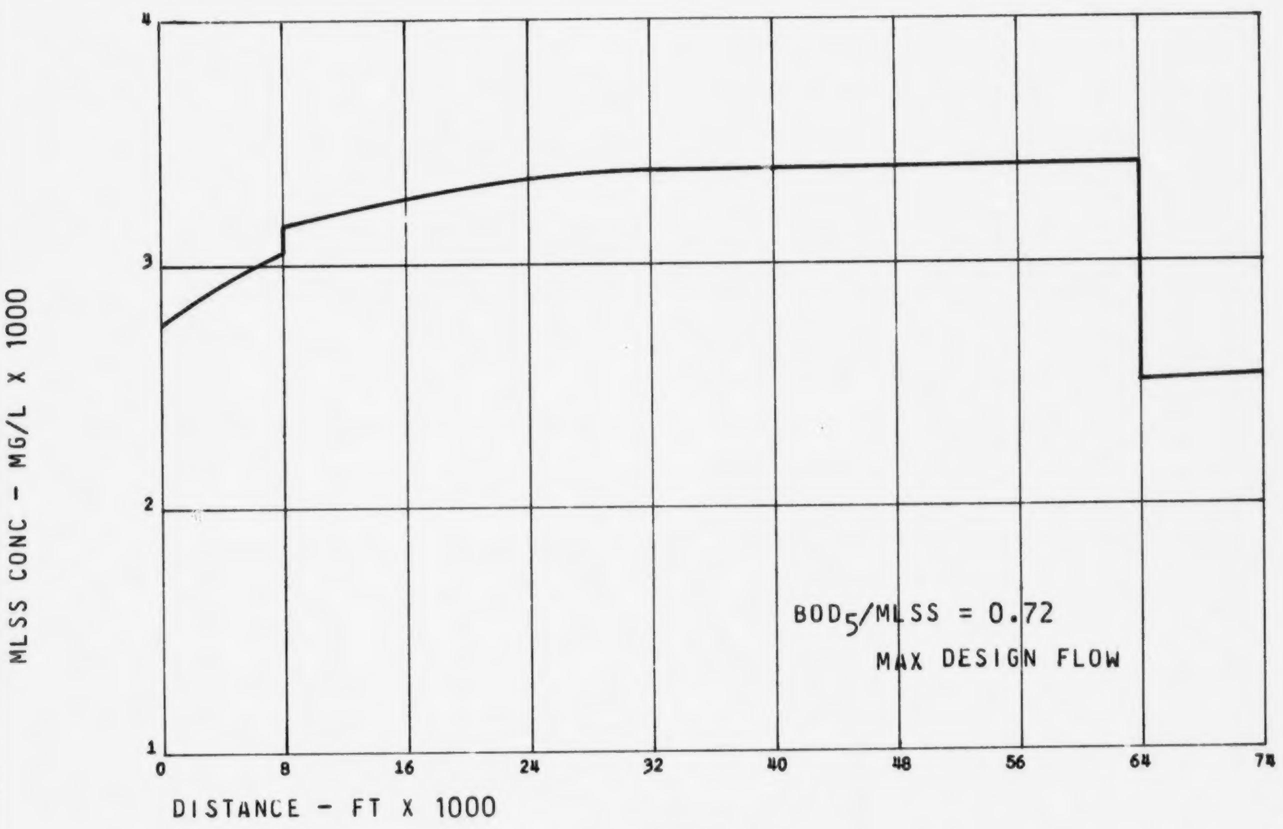


FIGURE 41
MLSS CONCENTRATION VS DISTANCE
ALTERNATE 3

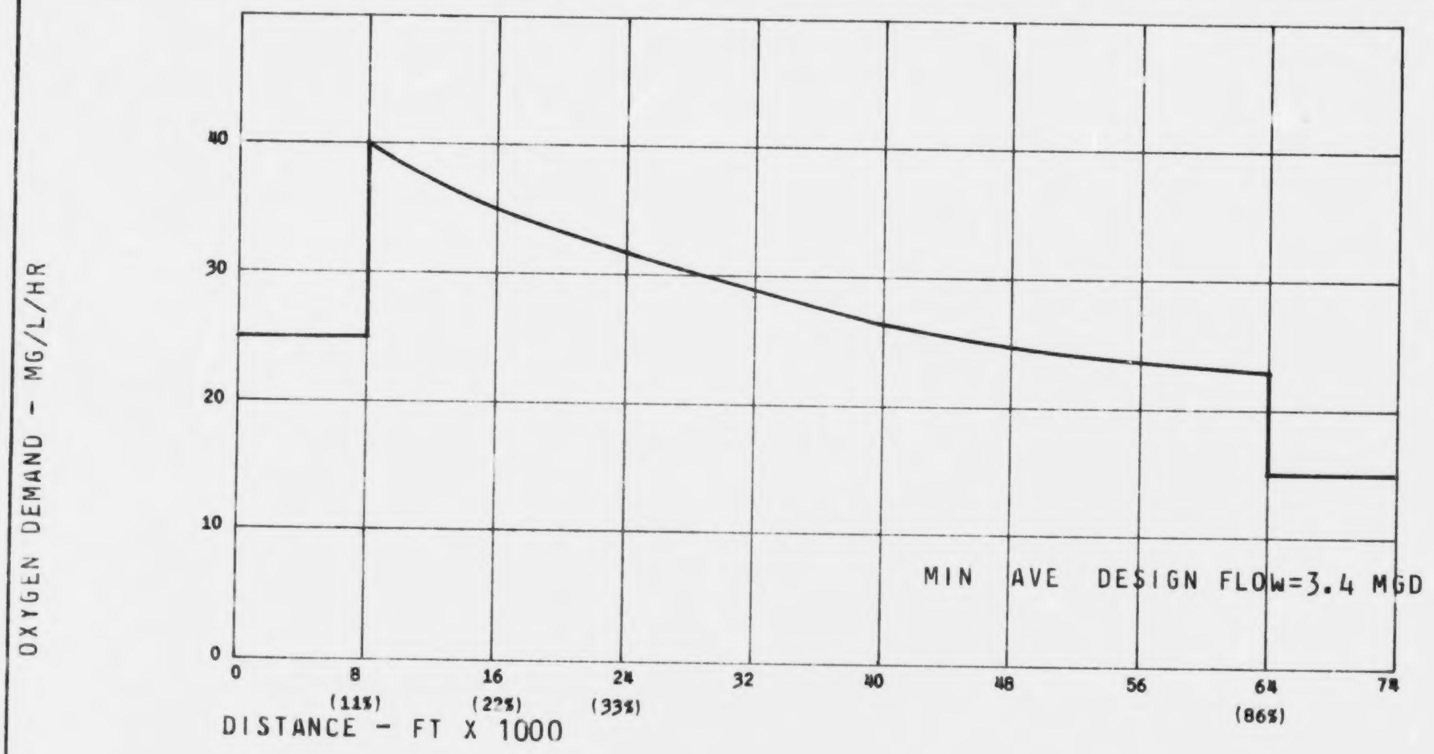


FIGURE 42
OXYGEN DEMAND VS DISTANCE
ALTERNATE 3

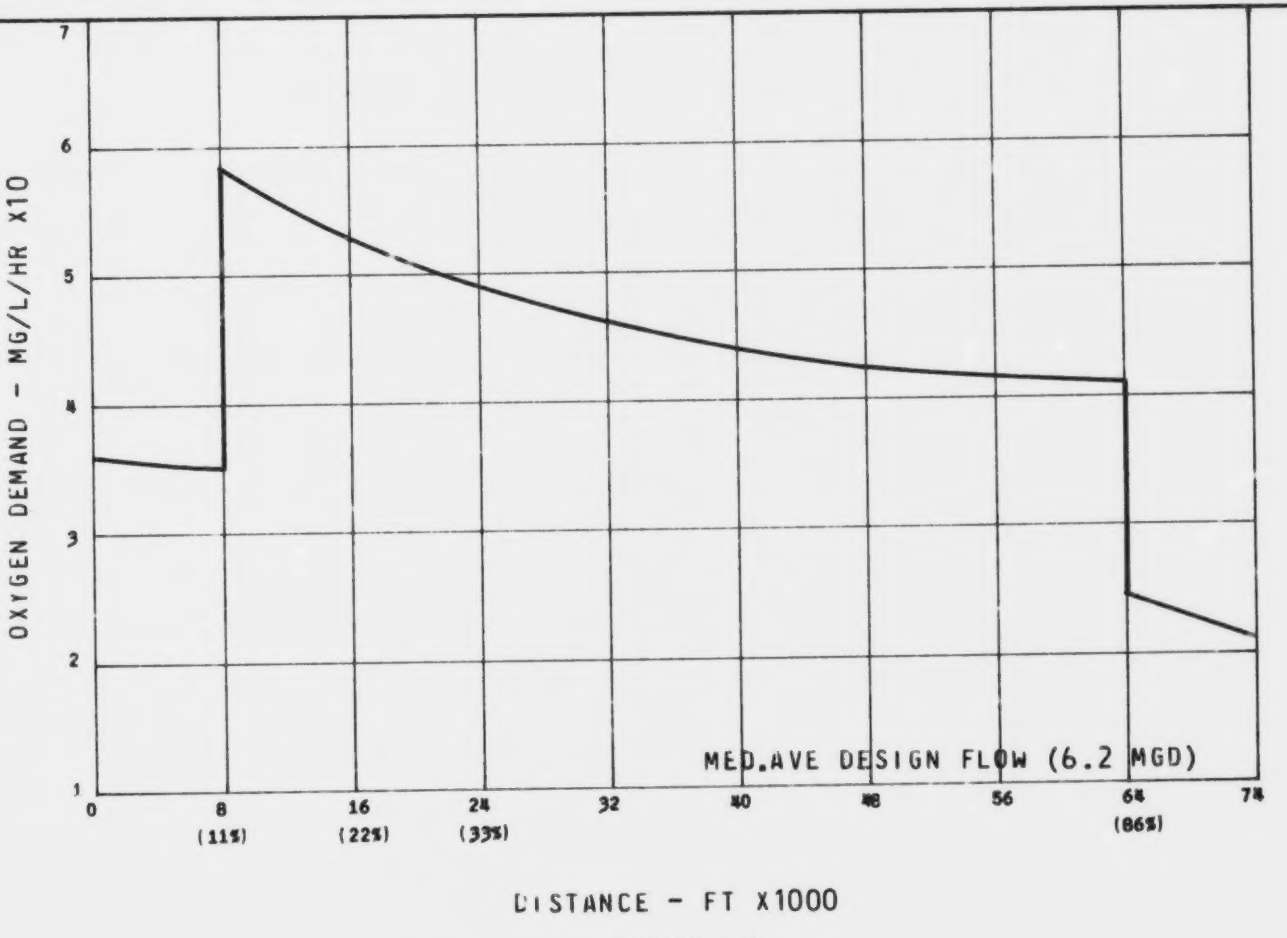


FIGURE 43
OXYGEN DEMAND VS DISTANCE
ALTERNATE 3

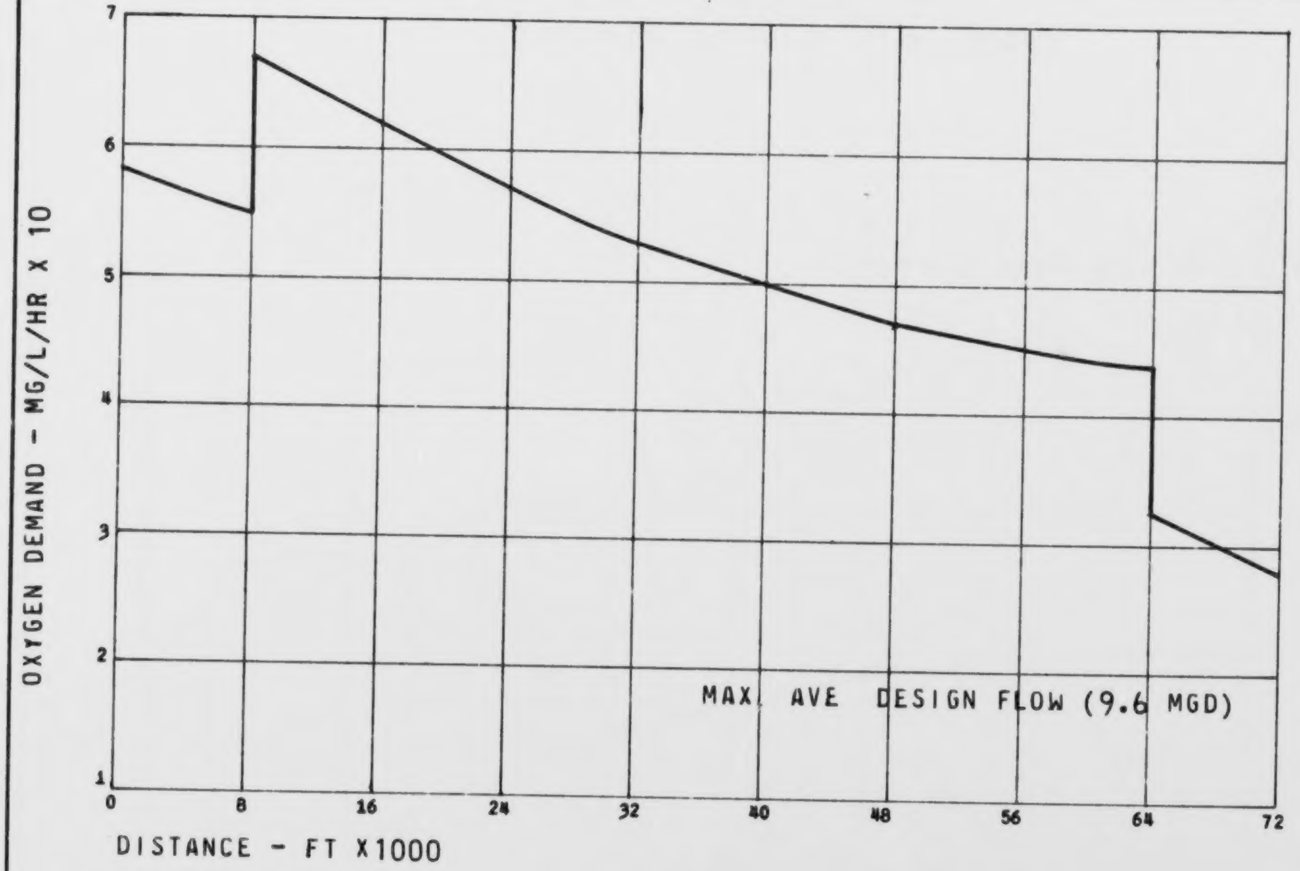


FIGURE 44
OXYGEN DEMAND VS DISTANCE
ALTERNATE 3

Tables 58, 59, and 60 show the aeration requirements in the channel for minimum, medium and maximum load conditions. These tables indicate the oxygen demand, diffuser spacing, number of blowers and blower capacity along the channel.

Tables 61, 62, and 63 show the blower capacity, horsepower, and air pipe sizing for each blower station along the channel system. The total system connected and operating horsepower is also tabulated on these tables.

The overall channel installation will require both air piping and return solids piping to be incorporated into the channel section, or laid parallel to the channel. A summary of these conduits sizes are indicated on Table 64.

DISCUSSION - Alternate 1 above proved to achieve relatively little BOD reduction due to the inability of the system to generate an active biological mass within the time restrictions. The cost of constructing the channel cannot be justified for the project under consideration unless significant BOD reductions can be achieved.

Alternate 2 above proved to achieve reasonably high BOD removals (70% to 80%) in the channel, providing an active biological mass is available. This biological mass could possibly be generated in an aeration system at the head of the channel, or it could be brought in. Neither of these methods appeared desirable for this particular study. Consideration was given to transporting the active biological mass generated at the Washburn plant to the head of the channel. Pumping of the activated sludge was evaluated and disgraded due to the technical problems inherent in pumping relatively small amounts of solids for long distances.

If large quantities of excess biological solids were available to insert at the head end of the channel, Alternate 2 could possibly provide the desired treatment efficiencies. While such solids are not readily available for the specific project under study, it could be applicable to other areas of the country. Before such a system is undertaken, however, detailed pilot plant studies should be undertaken to determine the detailed oxygen transfer and biological parameters.

Alternate 3, with the extensive sludge recycle system, would be the most practical approach to a biological system for the project under consideration. Even with this system, however, detailed pilot plant studies should be undertaken to determine detailed aeration and biological design parameters. The remaining discussion assumes selection of Alternate 3 treatment-transport system for the project under consideration.

The channel section, as previously discussed, would be of concrete construction with piping incorporated in the section. The design of the system included an evaluation of an open, or uncovered channel, versus a covered covering for flood protection as the top of the channel would be above the flood elevation of the Aroostook River, as discussed on Page 82.

TABLE 58

TOTAL AIR REQUIREMENTS

MINIMUM LOAD

ALTERNATE 3

Reach	Blower Location	Ave O_2 Demand(r_r) mg/1/hr	Depth of Flow(ft)	$(D)(r_r)$ $(C_L=1.0)$	D(ft)	Length (ft)	No of Diffusers	Blower Cap (CFM)
A to B	B	25	2.9	1,500	60	8,000	133	1,300
B to C	C	38	3.2	1,400	37	8,000	216	2,200
C to D	C	34	3.2	1,400	41.5	4,000	96	960
D to E	E	33	4.0	1,200	36.5	4,000	110	1,100
E to 28,000'	E	31	4.0	1,200	39	4,000	103	1,000
28,000' to F	F	29	4.0	1,200	41.5	4,000	96	1,000
F to 36,000	F	28	4.0	1,200	43	4,000	93	900
36,000 to G	G	27	4.0	1,200	44.5	4,000	90	900
G to 44,000	G	26	4.0	1,200	46	4,000	87	900
44,000 to H	H	25	4.0	1,200	48	4,000	83	850
H to 52,000	H	24	4.0	1,200	50	4,000	80	800
52,000 to I	I	23	4.0	1,200	52	4,000	77	800
I to 60,000	I	23	4.0	1,200	52	4,000	77	800
70,000 to J	J	23	4.0	1,200	52	4,000	77	800
J to 69,000	J	15	3.5	1,330	89	5,000	45	450
69,000 to K	K	15	3.5	1,330	89	5,000	45	450

TABLE 59

TOTAL AIR REQUIREMENTS

MEDIUM LOAD

ALTERNATE 3

Reach (ft)	Blower Location	Ave O_2 Demand(r_r) mg/1/hr	Depth of Flow(ft)	(D)(r_r) (C_L=1.0)	D(ft)	Channel Length(ft)	No of Diffusers	Blower Cap(CFM)
A to B	B	35	3.5	1,330	37	8,000	216	2,160
B to 12,000	B	57	4.7	1,080	19	4,000	210	2,100
12,000 to C	C	54	4.7	1,080	20	4,000	200	2,000
C to D	C	51	4.7	1,080	21.2	4,000	188	1,900
D to E	E	50	5.6	970	19.5	4,000	205	2,050
E to 28,000	E	48	5.6	970	20.2	4,000	198	2,000
28,000 to F	F	47	5.6	970	20.6	4,000	194	1,950
F to 36,000	F	46	5.6	970	21.1	4,000	190	1,900
36,000 to G	G	45	5.6	970	21.6	4,000	185	1,850
G to 44,000	G	44	5.6	970	22.0	4,000	182	1,800
44,000 to H	H	43	5.6	970	22.6	4,000	177	1,750
H to 52,000	H	42	5.6	970	23.1	4,000	173	1,750
52,000 to I	I	42	5.6	970	23.1	4,000	173	1,750
I to 60,000	I	41	5.6	970	23.6	4,000	170	1,700
60,000 to J	J	41	5.6	970	23.6	4,000	170	1,700
J to 69,000	J	24	5.0	1,040	43.5	5,000	115	1,150
69,000 to K	K	22	5.0	1,040	47.5	5,000	105	1,050

TABLE 60

TOTAL AIR REQUIREMENTS

MAXIMUM LOAD

ALTERNATE 3

Reach (ft)	Blower Location	Ave O_2 Demand(r_r) mg/1/hr	Depth of Flow(ft)	(D)(r_r) (C_L=1.0)	D(ft)	Channel Length(ft)	No of Difussers	Blower Cap(CFM)
A to B	B	57	3.7	1,280	22.5	8,000	355	3,600
B to 12,000	B	65	6.0	930	14.3	4,000	280	2,800
12,000 to C	C	63	6.0	930	14.8	4,000	270	2,700
C to D	C	60	6.0	930	15.5	4,000	258	2,600
D to E	E	58	6.9	850	14.7	4,000	272	2,700
E to 28,000	E	56	6.9	850	15.2	4,000	263	2,600
28,000 to F	F	54	6.9	850	15.8	4,000	253	2,500
F to 36,000	F	52	6.9	850	16.3	4,000	246	2,500
36,000 to G	G	50	6.9	850	17.0	4,000	235	2,300
G to 44,000	G	49	6.9	850	17.4	4,000	230	2,300
44,000 to H	H	48	6.9	850	17.7	4,000	226	2,300
H to 52,000	H	47	6.9	850	18.1	4,000	221	2,200
52,000 to I	I	46	6.9	850	18.5	4,000	216	2,200
I to 60,000	I	45	6.9	850	18.9	4,000	212	2,100
60,000 to J	J	45	6.9	850	19	4,000	212	2,100
J to 69,000	J	31	6.1	920	29.6	5,000	169	1,700
69,000 to K	K	29	6.1	920	31.8	5,000	157	1,600

TABLE 61

BLOWER AND AIR LINE SIZING

MINIMUM LOAD

ALTERNATE 3

Blower Location	Blower Cap (CFM)	Discharge psi	Blower hp	Air Line Length (ft)	Tentative Size (inch)	Reach Served (ft)
B	1,300	8	60	8,000	8 & 6	A to A
B	1,100	8	50	4,000	6	B to 12,000
C	1,100	8	50	4,000	6	12,000 to C
C	950	8	45	4,000	6	C to D
E	1,100	9	50	4,000	6	E to D
E	1,000	8	50	4,000	6	E to 28,000
F	1,000	8	50	4,000	6	F to 28,000
F	900	8	40	4,000	6	F to 36,000
G	900	8	40	4,000	6	G to 36,000
G	900	8	40	4,000	6	G to 44,000
H	850	8	40	4,000	6	H to 44,000
H	800	8	40	4,000	6	H to 52,000
I	800	8	40	4,000	6	I to 52,000
I	800	8	40	4,000	6	I to 52,000
J	800	8	40	4,000	6	J to 60,000
J	450	8	25	5,000	4 1/2	J to 69,000
K	450	8	25	5,000	4 1/2	K to 69,000
			725			

Total Connected HP

Blowers = 725
Return Pumps = 2(8)(40) = 640
Settling Tanks = 8(2) = 16
1,381

Total Operating HP

Blowers = 725
Return Pumps = (8)(40) = 320
Settling Tanks = 8(2) = 16
1,061

TABLE 62

BLOWER AND AIR LINE SIZING

MEDIUM LOAD

ALTERNATE 3

Blower Location	Blower Cap (CFM)	Discharge psi	Blower hp	Air Line Length (ft)	Tentative Size (inch)	Reach Served (ft)
B	2,200	8	100	8,000	12 & 8	B to A
B	2,100	8	100	4,000	10	B to 12,000
C	2,000	8	90	4,000	10	12,000 to C
C	1,900	8	90	4,000	8	C to D
E	2,050	8	100	4,000	10	E to D
E	2,000	8	100	4,000	10	E to 28,000
F	1,950	8	90	4,000	10	F to 28,000
F	1,900	8	90	4,000	8	F to 36,000
G	1,850	8	85	4,000	8	G to 36,000
G	1,800	8	85	4,000	8	G to 44,000
H	1,750	8	80	4,000	8	H to 44,000
H	1,750	8	80	4,000	8	H to 52,000
I	1,750	8	80	4,000	8	I to 52,000
I	1,700	8	80	4,000	8	I to 60,000
J	1,700	8	80	4,000	8	J to 60,000
J	1,150	8	55	5,000	6	J to 69,000
K	1,050	8	50	5,000	6	K to 69,000
			1,435			

Total HP	Connected	Operating
Blowers	1,435	1,435
Return Pumps	1,200	600
Set Tanks	16	16
	2,651	2,051

TABLE 63

BLOWER AND AIR LINE SIZING

MAXIMUM LOAD

ALTERNATE 3

Blower Location	Blower Cap(CFM)	Discharge psi	Blower hp	Air Line Length(ft)	Tentative Size(inch)	Reach Served(ft)
B	3,600	8	155	8,000	12 & 10	A to B
B	2,800	8	130	4,000	10	B to 12,000
C	2,700	8	125	4,000	10	C to 12,000
C	2,600	8	120	4,000	10	C to D
E	2,700	8	125	4,000	10	E to D
E	2,600	8	120	4,000	10	E to 28,000
F	2,500	8	115	4,000	10	F to 28,000
F	2,500	8	115	4,000	10	F to 36,000
G	2,300	8	105	4,000	10	G to 36,000
G	2,300	8	105	4,000	10	G to 44,000
H	2,300	8	105	4,000	10	H to 44,000
H	2,200	8	100	4,000	10	H to 52,000
I	2,200	8	100	4,000	10	I to 52,000
I	2,100	8	90	4,000	10	I to 60,000
J	2,100	8	90	4,000	10	J to 60,000
J	1,700	8	80	5,000	8	J to 69,000
K	1,600	8	75	5,000	8	K to 69,000
			1,855			

Total HP	Connected	Operating
Blowers	1,855	1,855
Sludge Return	1,200	600
Set Tanks	20	20
	3,075	2,475

TABLE 64

SUMMARY
CONDUIT PIPE SIZES

TREATMENT-TRANSPORT CHANNEL

ALTERNATE 3

Reach Served(ft)	Min Ave Flow				Ave Flow				Max Ave Flow			
	Air Piping		Sludge Return		Air Piping		Sludge Return		Air Piping		Sludge Return	
	Size (inch)	Length (ft)	Size (inch)	Length (ft)	Size (inch)	Length (ft)	Size (inch)	Length (ft)	Size (inch)	Length (ft)	Size (inch)	Length (ft)
A to 4,000	6	4,000	8	4,000	8	4,000	10	4,000	10	4,000	12	4,000
4,000 to B	8	4,000	8	4,000	12	4,000	10	4,000	12	4,000	12	4,000
B to 12,000	6	4,000	8	4,000	10	4,000	10	4,000	10	4,000	12	4,000
12,000 to C	6	4,000	8	4,000	10	4,000	10	4,000	10	4,000	12	4,000
C to D	6	4,000	8	4,000	8	4,000	10	4,000	10	4,000	12	4,000
D to E	6	4,000	8	4,000	10	4,000	10	4,000	10	4,000	12	4,000
E to 28,000	6	4,000	8	4,000	10	4,000	10	4,000	10	4,000	12	4,000
28,000 to F	6	4,000	8	4,000	10	4,000	10	4,000	10	4,000	12	4,000
F to 36,000	6	4,000	8	4,000	8	4,000	10	4,000	10	4,000	12	4,000
36,000 to 6	6	4,000	8	4,000	8	4,000	10	4,000	10	4,000	12	4,000
G to 44,000	6	4,000	8	4,000	8	4,000	10	4,000	10	4,000	12	4,000
44,000 to H	6	4,000	8	4,000	8	4,000	10	4,000	10	4,000	12	4,000
H to 52,000	6	4,000	8	4,000	8	4,000	10	4,000	10	4,000	12	4,000
52,000 to I	6	4,000	8	4,000	8	4,000	10	4,000	10	4,000	12	4,000
I to 60,000	6	4,000	8	4,000	8	4,000	10	4,000	10	4,000	12	4,000
60,000 to J	6	4,000	8	4,000	8	4,000	10	4,000	10	4,000	12	4,000
J to 69,000	4 1/2	5,000	8	5,000	6	5,000	10	5,000	8	5,000	12	5,000
69,000 to K	4 1/2	5,000	8	5,000	6	5,000	10	5,000	8	5,000	12	5,000

NOTE: From "B" to "J", there are two (2) sludge return pipes of equal size.

Cold weather, however, could seriously affect channel performance. The Aroostook River Basin experiences extended winter periods when the temperatures do not rise above 0°F, and frequently dip to -30°F to -40°F. In addition, because of the relatively flat topography, blowing and drifting snow creates many problems. With an uncovered channel system, severe icing and frozen mist problems would probably reduce treatment efficiencies. Open exposure of the channel would also reduce the effectiveness of the treatment process through radiational cooling. The channel could also become partially or fully clogged with drifting snow. Because of the potential winter problems with an open channel system, it is recommended that the channel be covered. In addition, if an open channel system is used, both sides of the channel would have to be fenced for safety reasons. This would be extremely expensive, and would severely limit access to the properties abutting the channel route.

The channel will be laid in trench in most areas with some embankment sections. In areas where significant brooks must be crossed, the section may be converted to a standard pipe section of convenience. Care must be taken to provide an adequate surface water drainage system from the uphill drainage areas, and to assure protection of the adjacent railroad grades. While this will require special attention during design, no extraodinary problems are anticipated.

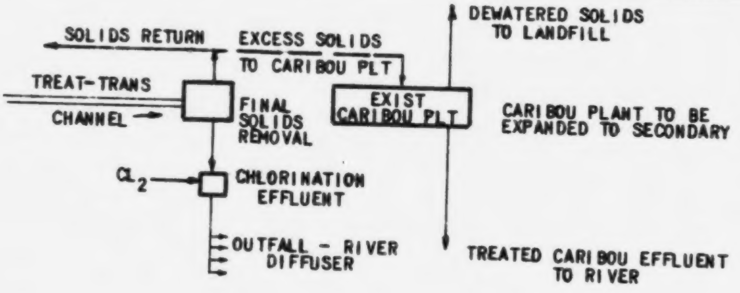
The locations for solids return pumps, blowers etc are generally quite accessible from existing roads, and should present few site problems. Electric power must be available at each location. With the power requirements at each, it is expected that the power company can easily justify providing such service.

While the overall treatment-transport channel concept is rather unique, it appears to be technically feasible with Alternative 3 to achieve reasonable BOD removals in the system. The cost of such a system and its relation to other alternatives is discussed in a later section.

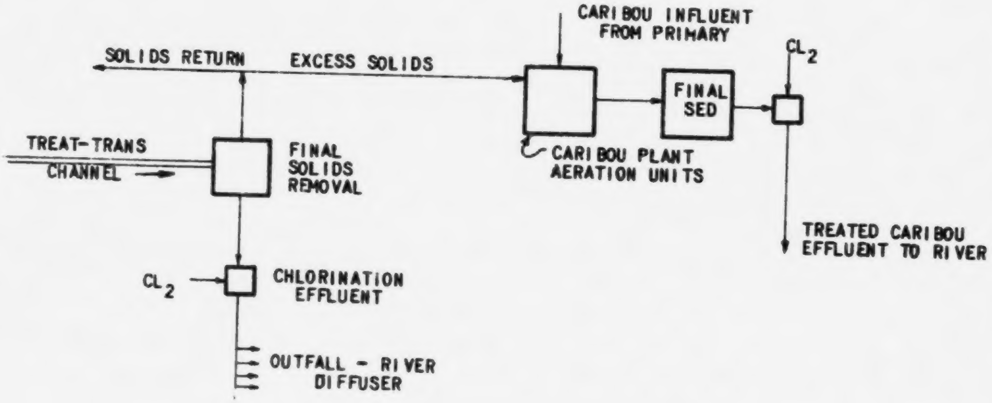
Upon discharge from the treatment-transport channel, the mixed liquor must be treated in a terminal facility prior to final disposal. These terminal facilities must be coordinated with the expansion of the Caribou facilities. The alternatives studied for these facilities are briefly described as follows:

1. Chlorination of the clarified channel effluent with discharge to the Aroostook River at Caribou. The solids generated in the channel system would be dewatered with expanded solids handling facilities at the Caribou in-town plant. In addition, the Caribou plant would treat the Caribou wastes in the same manner as described under community system.

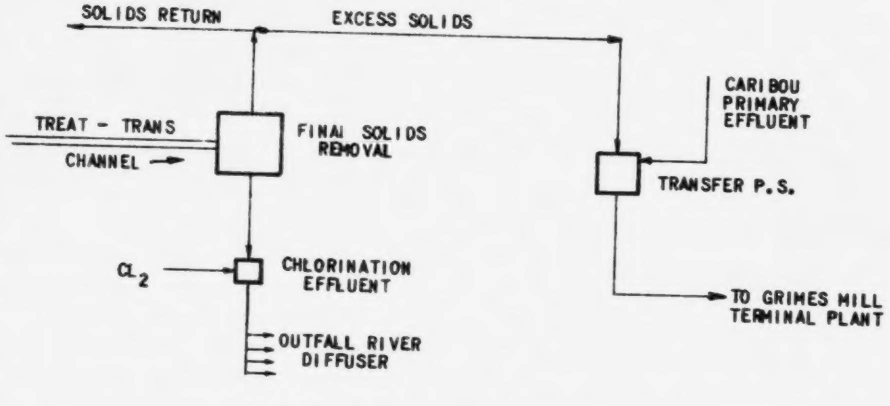
2. Chlorination of the clarified channel effluent with discharge to the Aroostook River at Caribou. The excess biological solids from the channel system would be used in the Caribou in-town aeration tanks for contact stabilization of the incoming Caribou wastes.


DEWATERED SOLIDS
TO LANDFILL
SOLIDS RETURN
EXCESS SOLIDS
TO CARIBOU PLT
TREAT-TRANS
CHANNEL
FINAL
SOLIDS
REMOVAL
EXIST
CARIBOU PLT
CARIBOU PLANT TO BE
EXPANDED TO SECONDARY
CL2
CHLORINATION
EFFLUENT
OUTFALL - RIVER
DIFFUSER
TREATED CARIBOU EFFLUENT
TO RIVER

TERMINAL FACILITY 1


CARIBOU INFLUENT
FROM PRIMARY
SOLIDS RETURN
EXCESS SOLIDS
CL2
FINAL
SED
TREAT-TRANS
CHANNEL
FINAL
SOLIDS
REMOVAL
CARIBOU PLANT
AERATION UNITS
TREATED CARIBOU
EFFLUENT TO RIVER
CL2
CHLORINATION
EFFLUENT
OUTFALL - RIVER
DIFFUSER

TERMINAL FACILITY 2


SOLIDS RETURN
EXCESS SOLIDS
CARIBOU
PRIMARY
EFFLUENT
TREAT - TRANS
CHANNEL
FINAL SOLIDS
REMOVAL
TRANSFER P.S.
CL2
CHLORINATION
EFFLUENT
TO GRIMES MILL
TERMINAL PLANT
OUTFALL RIVER
DIFFUSER

TERMINAL FACILITY 3

FIGURE45
SCHEMATIC TREATMENT-TRANSPORT
TERMINAL FACILITIES

3. Chlorination of the clarified channel effluent with discharge to the Aroostook River at Caribou. The excess biological solids from the channel system would be combined with the incoming Caribou wastes for transport to Grimes Mill for final processing.

The schematic configurations of the three terminal alternatives are shown on Figure 45. In all three cases the channel system effluent would be chlorinated and discharged to the river at Caribou. The chlorination facilities would be an expansion of those at the in-town Caribou plant. The first terminal alternative assumes no attempt to utilize the excess biological mass generated in the channel system for contact stabilization with the Caribou wastes. In this case the in-town Caribou plant would treat the incoming Caribou wastes, as described for the community in-town plant expansion, except that the solids handling facilities would be expanded. This terminal alternative is not adaptable if the Caribou wastes are to be transferred to Grimes Mill for treatment.

The second terminal alternative again considers that the Caribou wastewaters will be treated by expansion of the in-town plant, and not transferred to Grimes Mill. Again the channel effluent would be chlorinated and discharged to the river. The excess biological mass would be entered into the Caribou plant aeration facilities. If this is accomplished, a contact stabilization system will in a sense be created. Table 65 indicates the BOD to MLSS ratios which will be created in the system upon mixing.

TABLE 65

BOD TO MLSS RATIO - CARIBOU PLANT AERATION

	Flow Condition (MGD)		Conduit Waste	Caribou BOD_5	BOD_5 to MLSS
	Conduit	Caribou	Sol lb/day	lb/day	Ratio
Min Load	3.4	2.7	9,800	19,400	0.20
Med Load	6.2	3.0	34,500	26,400	0.76
Max Load	9.6	4.6	49,000	54,000	1.10

A desirable BOD to MLSS ratio would be from about 0.4 to 0.8. Thus it can be seen that except for the minium flow conditions, a reasonable mix can be obtained. Limited studies by Atkins and Sproul[9] and Cornell, Howland, Hayes and Merryfield[20] indicate reasonable BOD removals may be achieved by contact stabilization techniques. BOD removals of about 80% were reported with a one hour contact time. As new active solids would be continually supplied by the channel system solids, reaeration and return would not be required, or would require minimum facilities. Ideally the mixed liquor leaving the contact aeration tanks could be clarified with the effluent, chlorinated and discharged, and the solids thickened and dewatered.

It must be recognized, however, that reliable design data for such a system is not currently available. Prior to final design, considerable laboratory, and possibly pilot plant work, would be required to establish a reasonable contact time, and to refine removal efficiencies. For preliminary analysis a 4 to 6 hour aeration-contact time was adopted, recognizing that the system does not represent a true contact stabilization plant in the theoretical sense. Thus, the aeration volume capacity for the in-town Caribou plant may be reduced substantially if the contact stabilization concept is adopted and is successful. The final clarifiers at the plant will not change greatly regardless of the type of biological system used.

The third terminal alternate is similar to the second one, except the mixed Caribou wastewaters and the channel excess solids will be transported to Grimes Mill for final treatment. The in-transit time between Caribou and Grimes Mill will be about 1 hour. If a one hour contact time were sufficient to achieve full treatment of the Caribou wastes, the transit system could serve as a contact mechanism. However, current data is not sufficient to plan on such efficiency, although more detailed laboratory studies may give it some credence. For preliminary analysis it has been assumed that a 6 to 8 hour terminal aeration time will be provided at Grimes Mill for the mixed liquor and any incoming flows from the Grimes Mill industrial park.

The plant at Grimes Mill would be similar in orientation and construction to that illustrated in subsequent sections for the local community systems, except for some reduction in aeration volume and an increase in solids handling facilities.

The Alternate 3 treatment-transport channel system combined with terminal facilities at either Caribou or Grimes Mill, could provide a treatment system capable of handling the design loads from the Core Area in a manner that will meet current water quality standards on both the Prestile Stream and Aroostook River.

SECTION VII

REGIONAL TREATMENT ANALYSES

Preliminary designs have been prepared for facilities to treat the range of waste loads generated within the region, as presented in the previous section. The preliminary design studies considered numerous system alternatives, in addition to the various loading options. The system designs were subdivided into three major categories; namely, individual community treatment facilities, partially integrated or interconnected regional facilities, and fully integrated regional facilities. Through this mechanism it is possible to determine the system best suited to the region as a whole, and each community within the region.

The most common approach to the pollution abatement problems of the Basin would be to construct separate treatment facilities for each significant source of pollution. That is, at each community and at each industry. With this approach, as many as 18 treatment plants could be operated in the Aroostook-Prestile Basins. A modification of the individual approach would be the construction of joint industry-municipal plants in each community. This would provide significant consolidation of waste flows, but would still result in a multi-plant system.

In addition to local industry-municipal consolidation, varying degrees of intercommunity interconnection are also possible. This type of consolidation could range from complete interconnection from Washburn to Fort Fairfield, to a simple two community joint project. The analyses of this section, and Section VIII, consider all reasonably viable combinations. Evaluation of these alternatives must be made in light of technical feasibility, relative total costs, and impact on water quality. The following subsections discuss the preliminary designs and technical aspects of the facility options. The financial implications of each proposed system are presented in subsequent sections.

The geographical orientation of the communities and industrial waste sources reveal that the region can be subdivided into a "Core Area" encompassing Presque Isle, Easton and Caribou; excluding outlying communities such as Washburn, Mapleton and Fort Fairfield. The geographical "Core Area" is frequently referred to in the following text.

SYSTEM OPTIMIZATION - The primary objective of the project was to determine the optimum system for treatment and disposal of all wastewaters being discharged to the Aroostook River and Prestile Stream. The proposal for the project outlined linear programming techniques to be used for determining the optimum configuration of the proposed treatment-transport system. The programming format, outlined in highly simplified terms in the proposal, is as follows:

An objective function would be written which would contain all the factors which would influence the total cost of waste treatment, subject to pilot plant constraints.

$$Y = c_1 x_1 + c_2 x_2 + \dots\dots\dots c_n x_n$$

where Y = total cost

$x_1, x_2 \dots x_n$ = individual cost factors (such as cost of aeration, construction, etc)

$c_1, c_2 \dots c_n$ = unit cost of each cost factor

An example of total cost factor is the cost of aeration. (Let this be represented by x_1.) The amount of oxygen which must be transferred to the mixed liquor for a given number and spacing of aeration units (assuming use of the treatment-transport system) can be calculated from the results of the pilot plant data. (x_1 assumes this value.) c_1, then, is the cost of transferring one pound of air. If, for a system arrangement, requiring the transfer of 1,000 pounds of oxygen per hour, and the unit cost is \$5 per pound of oxygen transferred per hour, the first term in the objective function becomes:

$$c_1 x_1 = 5.00 \ (1{,}000)$$

or

$$Y = 5.00 \ (1{,}000) + c_2 x_2 + \dots\dots\dots c_n x_n$$

If x_2 is the number of aeration units required to transfer 1,000 pounds of oxygen to the treatment-transport system (say $x_2 = 20$) and the unit cost for construction of an aeration unit is \$700, the second term of the objective function becomes $c_2 x_2 = 700 \ (20)$ and the objective function cannot be written as:

$$Y = 5.00 \ (1{,}000) + 7.00 \ (20) + c_3 x_3 + c_n x_n$$

The process is continued until all factors influencing the cost of treatment are included in the objective function.

A sensitivity analysis would be carried out to determine which cost factors can be ignored as having insignificant response to the overall cost. Thus, the smallest objective function can be developed which can be handled easily. If such an analysis yields five factors of significant sensitivity ($n = 5$), then the (general) objective function becomes:

$$Y = c_1 x_1 + c_2 x_2 + c_3 x_3 + c_4 x_4 + c_5 x_5$$

When this has been accomplished, the cost factors (x_1, x_2, x_3, x_4, x_5) can be changed systematically (using a computer) to determine the cost for a very large number of different systems. Some of the terms of the objective function will increase under the influence of a given change, while others will decrease. In this way, an optimum solution can be determined.

The use of optimization techniques for establishing an optimum treatment system is contingent on the availability of parameters for each variable function. The original proposal included pilot plant studies to determine such parameters. This portion of the project, however, was eliminated by EPA on the assumption that they could provide the necessary data from existing EPA projects.

Initial project concept was to use optimization techniques for the design of the treatment-transport system. Such techniques would be used to determine aerator spacing, solids return volumes, location of sludge return structures, etc. The detailed parameters required to justify such techniques, however, were not available from published literature, or from the information provided by EPA. Without the pilot plant portion of the project, the data could not be generated.

As discussed in Section VI, the data available for designing the treatment-transport system was drawn from sources that were not directly related to the system in question. Many assumptions were required, therefore, in developing the basic design parameters for the treatment-transport system.

The design parameters developed could have been used in conjunction with optimization techniques, but serious difficulties could result, as follows:

1. The data developed was not of the desired sensitivity to accurately determine optimum design recommendations.
2. Because of the nature of the data, sensitivity analysis of the various factors could be very misleading.
3. Future researchers could be mislead if the results of such optimization studies were used without full knowledge of the data generation.

Because of the inherent dangers, as outlined above, associated with optimization procedures based on only general data, it was decided not to use this technique even though it was outlined in the initial proposal. If the pilot plant work had been undertaken, or if EPA had provided sufficient data, as indicated, optimization techniques for determining the optimum configuration of the treatment-transport system would have been employed.

DESIGN CONDITION CODING - Design conditions for the majority of the communities included in this study consist of both domestic and industrial waste loads. As presented in Section V, Tables 28 through 39, the residual waste loads from each industry could vary considerably, depending on the degree of in-plant process modification. A coding summary of design conditions for treatment facility design is presented for each system evaluated.

INDIVIDUAL COMMUNITY SYSTEMS

WASHBURN AREA - Preliminary designs were prepared for the Washburn area waste loads, as presented in Tables 28, 29, and 30. Designs include individual, industrial and domestic facilities, and the option of providing improved sedimentation facilities at Taterstate, or retaining the existing unit.

The question of joint versus separate treatment of domestic and industrial wastewaters is complex. Several factors must be considered; including capital costs, operating costs, technical compatibility, governmental aid policy, and local administrative policy. Planning to date at Washburn has been based on construction of a single plant to treat both the domestic and industrial wastewaters. This policy has been reviewed and updated in light of current technology and administrative factors.

The interrelation of the domestic wastewaters of the Town and the processing wastewaters of the Taterstate plant becomes quite complex when the variable factor of in-plant adjustments at Taterstate is considered. It is further complicated by the option of improved primary treatment and potential product expansion at Taterstate. Figure 46 illustrates the basic system of wastewater flow at Washburn, with the alternates available. The basic alternate is connection, or non-connection, of the municipal wastes with the industrial wastes. If not connected, two treatment plants will result or, if connected, a single plant. The Taterstate plant has the options of in-plant changes as disucssed in the previous section. It also has the option of retaining its existing sedimentation-flotation primary treatment facility, or replacing it with a gravity sedimentation facility.

The existing sedimentation-flotation unit is removing only 10% to 20% of the plant BOD, and about 20% to 35% of the plant suspended solids. This unit is considerably less efficient than a well designed gravity sedimentation unit. A gravity unit would be expected to remove 40% to 50% BOD and up to 85% of the suspended solids. The existing unit is designed to function as an air flotation unit. Its configuration and equipment, however, is such that it is essentially operating as a sedimentation unit. It is overloaded for this use, and its solids removal equipment is inadequate for the job intended. The solids retention time of this unit leads to fermentation and a lowering of effluent PH to below allowable standards. This unit is not adequate for primary solids removal, and should be taken out of the active service when replaced by an efficient gravity sedimentation unit. All subsequent studies assume replacement of this unit.

It is also understood that management is considering product expansion in the form of a potato flake line at the recently acquired Staley Building. As many as 90 tons of potatoes per day could be processed through this line, resulting in waste loads as previously presented in Table 30. Thus, preliminary design analyses have considered treatment facilities with, and without, this potential expansion.

The many variables in load conditions at Washburn create a great number of potential problem solutions. The studied combinations are coded as indicated in Table 66.

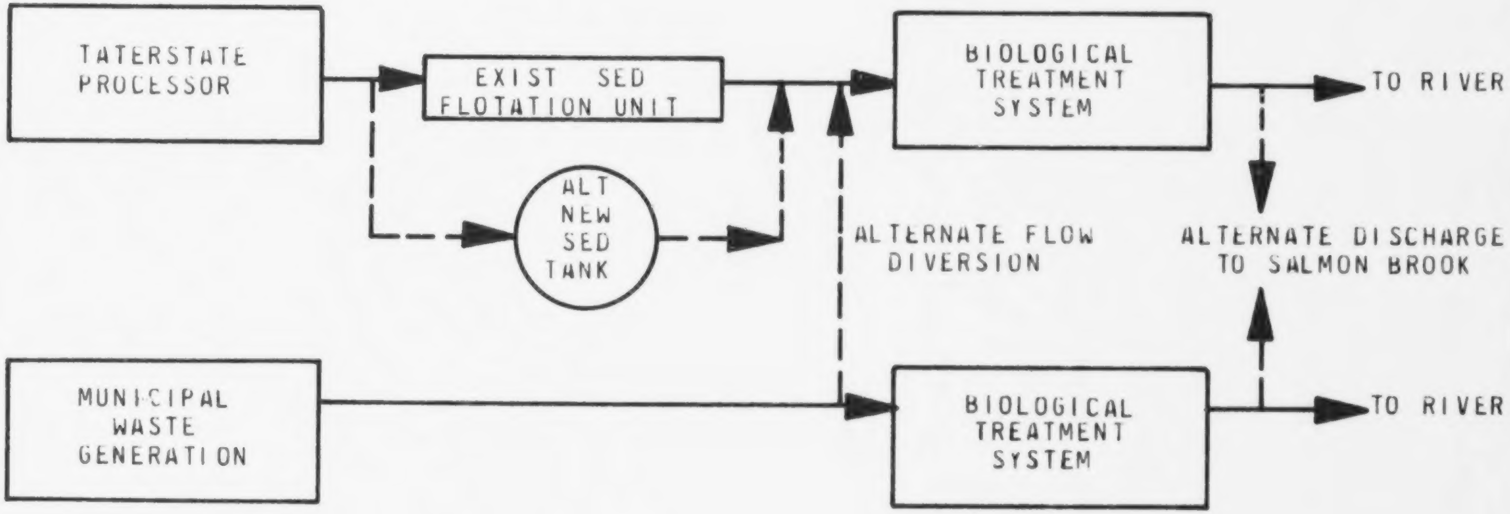

TATERSTATE PROCESSOR
EXIST SED FLOTATION UNIT
ALT NEW SED TANK
BIOLOGICAL TREATMENT SYSTEM
TO RIVER
ALTERNATE FLOW DIVERSION
ALTERNATE DISCHARGE TO SALMON BROOK
MUNICIPAL WASTE GENERATION
BIOLOGICAL TREATMENT SYSTEM
TO RIVER

FIGURE 46

DESIGN CONDITIONS- WASHBURN-SCHEMATIC

TABLE 66

DESIGN CONDITION - CODING SUMMARY

WASHBURN SYSTEM

"A" Conditions - Loading Alternatives at Taterstate

A-1 - Operation during Spring of 1971 - no in-plant change
A-2 - In-plant flow reductions and screen change
A-3 - In-plant flow reduction, plus dry caustic peel conversion

"B" Conditions - Product Expansion Alternatives

B-1 - Retain existing production capacity
B-2 - Expand with new flake line

"C" Conditions - Municipal-Industrial Coordination

C-1 - Taterstate without municipal wastes in following combinations of "A" & "B" conditions:

A-1 + B-1	A-2 + B-1	A-3 + B-1
A-1 + B-2	A-2 + B-2	A-3 + B-2

C-2 - Taterstate combined with municipal wastes in following combinations of "A" & "B" conditions:

A-1 + B-1	A-2 + B-1	A-3 + B-1
A-1 + B-2	A-2 + B-2	A-3 + B-2

C-3 - Municipal wastewaters alone.

The loads to the treatment plant for "A" conditions, reflecting alternate in-plant conditions at Taterstate and the impact of flake line expansion are as previously presented in Tables 29 and 30. Preliminary designs of all combinations outlined in the foregoing paragraphs were accomplished by means of a computer design program developed by the Edward C. Jordan Company, Consultants to the Commission. Such computer techniques were mandatory to evaluate all possible combinations of waste loads, and to study the effect of varying design criteria. The detailed computer design program is presented in Section XIII.

In addition to the previously outlined load conditions, the preliminary designs reflect both single unit and dual unit construction. In publicly owned plants constructed with government grants-in-aid, dual units of most plant components are required. In many industrial plants, however, only a single sequence of units is provided. The use of dual units provides greater operational flexibility, and provides some degree of redundancy. Dual units, of course, cost more than single units. For full evaluation of a joint domestic-industrial treatment facility, data on both systems is necessary.

For all treatment systems evaluated a single primary settling tank was included for the industrial wastewater. This unit will treat only industrial wastewater, and can be taken out of service annually for inspection and repair. The existing sedimentation-flotation unit will also provide some degree of standby potential. In all cases where joint treatment is considered, dual components have been provided. This will be necessary for consideration of governmental grants-in-aid.

Certain key equipment in all configurations, including strictly industrial usage, would be provided in at least two units. These include chlorinators, vacuum filters, aerators, and solids return pumps.

The preliminary design sizing of key treatment plant units is shown in Tables 67, 68 and 69.

Review of the foregoing tables suggest the following:

1. A considerable reduction in plant unit sizing can be achieved if the Taterstate plant converts to the dry caustic peel system, providing the system can achieve the estimated waste load reductions.
2. The increased size requirement to handle the municipal wastes are relatively significant in the units with hydraulic design criteria such as aeration tanks, sedimentation tanks, etc., but are relatively insignificant on the organic design criteria items.
3. The aeration volume required for the municpal wastes alone are nearly as great, or greater than, the volumes required for combined waste treatment. This is due to the long aeration time of the municipal extended aeration system. For the municipal plant alone, this is necessary to effectively handle the small volumes of excess solids generated.
4. Based on the data presented above, serious consideration should be given to joint treatment of the domestic and industrial wastewaters.

Preliminary designs indicate that combined treatment has advantages over separate treatment. The cost impact of such combination is discussed in the next section. The topography of the Town of Washburn is such that all wastes, both domestic and industrial are concentrated in the vicinity of the Taterstate plant. No additional transportation costs are required to place the wastewaters in a combined plant as opposed to individual plants. Thus, any plants, or combination of plants, considered at Washburn will be located adjacent to the Wade Road on the westerly side of Salmon Brook, or on the higher ground northerly of Wade Road. Either location would appear suited and final selection should be made during detailed design.

TABLE 67

KEY TREATMENT PLANT SIZING - WASHBURN

CONDITION C-1 TATERSTATE PLANT ALONE - TOTAL UNIT SIZE*

Sub Condition	Primary Sed Tank Dia-ft	Aeration Tank Vol CF	Secondary Sed Tank Surf Area	O_2 Req Lbs/hr	Excess Solids lbs/day	Vac Filter A-Sq ft	CL_2 Contact Vol CF	CL_2 Req lbs/day
A-1 + B-1	46	86,968	2,169	573	29,100	366	1,400	85
A-2 + B-1	42	73,829	1,841	486	24,700	311	1,183	75
A-3 + B-1	36	52,181	1,301	241	10,000	134	837	55
A-1 + B-2	52	110,000	2,752	728	37,000	465	1,769	110
A-2 + B-2	48	95,602	2,384	630	32,100	403	1,532	100
A-3 + B-2	40	66,821	1,666	308	12,800	172	1,071	70

*For Dual System - Primary tank as given, subdivide other units as appropriate to obtain sizing of each unit.

NOTE: See Table 66 for design condition coding summary.

TABLE 68

KEY TREATMENT PLANT SIZING - WASHBURN

CONDITION C-2 TATERSTATE AND DOMESTIC WASTES COMBINED - TOTAL UNIT SIZE*

Sub Condition	Primary Sed Tank Dia-Ft	Aeration Tank Vol CF	Secondary Sed Tank Surf Area	O_2 Req lbs/hr	Excess Solids lbs/day	Vac Filter A-Sq ft	CL_2 Contact Vol CF	CL_2 Req lbs/day
A-1 + B-1	46	116,624	2,908	594	29,300	374	1,870	115
A-2 + B-1	42	103,362	2,578	502	24,700	313	1,658	100
A-3 + B-1	36	81,838	2,042	246	9,800	130	1,312	80
A-1 + B-2	52	139,900	3,488	744	37,000	470	2,242	135
A-2 + B-2	48	125,134	3,120	646	32,100	410	2,060	120
A-3 + B-2	40	96,354	2,402	324	12,800	175	1,544	95

*For Dual System - Primary tank as given, subdivide other units as appropriate to obtain sizing of each unit.

NOTE: See Table 66 for design condition coding summary.

TABLE 69

KEY TREATMENT PLANT SIZING - WASHBURN

CONDITION C-3 MUNICIPAL WASTES ALONE - TOTAL SIZING*

Sub Condition	Primary Sed Tank Dia-ft	Aeration Tank Vol CF	Secondary Sed Tank Surf Area	O_2 Req lbs/hr	Excess Solids lbs/day	Vac Filter A-Sq ft	CL_2 Contact Vol CF	CL_2 Req lbs/day
24 hr Aer	None	90,866	413	15	60	NA	474	29
20 hr Aer	None	75,600	413	15	80	NA	474	29

* For Dual Units - Divide appropriate elements by 2.

NOTE: See Table 66 for design condition coding summary.

Preliminary designs were based on the use of concrete tanks for primary and secondary sedimentation, and the chlorine contact tank. The aeration basins were assumed to be of earthen embankment construction with a concrete lining. Fixed platform mechanical aerators were utilized. Air flotation was used for excess activated sludge thickening prior to vacuum filtration. The sedimentation tanks and aeration tanks were left uncovered. Other equipment would be enclosed in the control building. All wastewaters will be pumped to the plant. It will be necessary to raise the lower plant site grade for flood protection, if used, or to lift to the higher plant site. Figure 47 shows a typical site plan for the lower site adjacent to Salmon Brook. The basic configuration would remain the same for all loading conditions, with only the sizing varied as presented in the previous tables. The configuration for the upper site, if used, would be similar although specific adaptation to the site will be required.

Hydrologic studies presented in the companion Basin Planning Report indicate that the flow from Salmon Brook does not mix well with the Aroostook River flow. The Salmon Brook tends to stratify horizontally and follow the north and east shore of the Aroostook River for a considerable distance downstream. This, combined with the limited natural flow in Salmon Brook, restricts waste load input to the brook. Construction of a piped outfall to the river with a diffuser system to disperse the treated wastewater in the full available flow of the river is recommended. If a municipal plant is constructed for the domestic wastewaters only, direct discharge to Salmon Brook would appear adequate.

MAPLETON AREA - Expanded water pollution control facilities will be required in the Mapleton area in the relatively near future. The Mapleton Sewer District proposes to construct a treatment plant to handle the domestic wastewaters, with discharge to the North Branch of the Presque Isle Stream. The plant will be of the extended aeration type with facilities for excess solids handling. The entire facility will be enclosed for cold weather protection.

The proposed Mapleton facilities have been reviewed and are considered quite adequate to meet the domestic needs of the community. No appreciable amount of industrial wastewater is anticipated in the Mapleton area.

PRESQUE ISLE AREA - Preliminary designs were prepared for the expansion of the existing primary plant of the Presque Isle Sewer District to meet current water quality standards. Likewise preliminary designs were prepared for provision of effective secondary treatment at Potato Service Inc. A further design considered consolidation of all wastewaters within the City at a single treatment facility. Each of these options is discussed in the following paragraphs. The load conditions at Presque Isle provide several potential problem solutions. The studied combinations are coded as indicated in Table 70.

The existing Presque Isle municipal plant actually consists of two elements. The main element is a primary treatment facility serving the

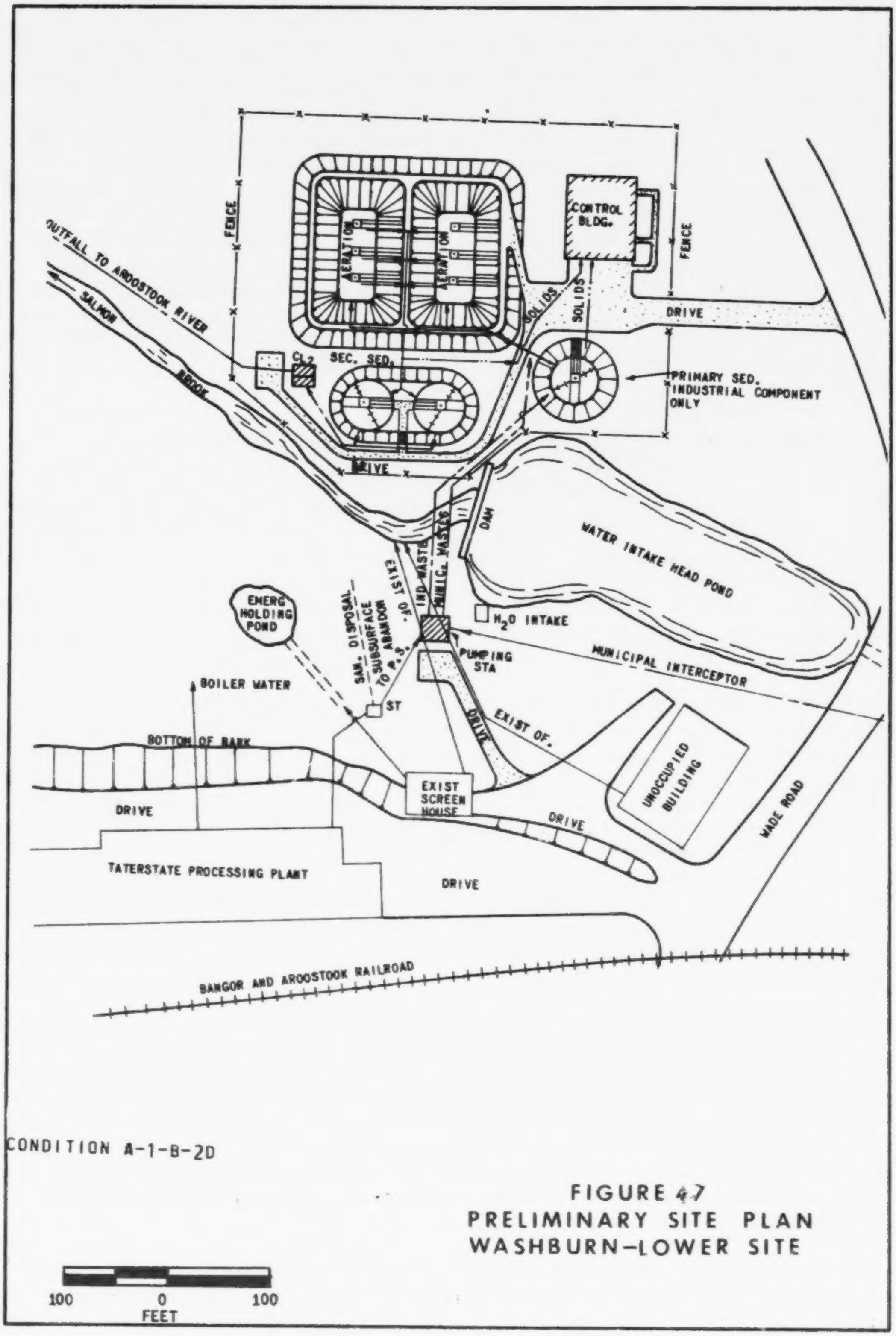

FENCE
FENCE
OUTFALL TO AROOSTOOK RIVER
SALMON
BROOK
AERATION
AERATION
CONTROL BLDG.
SOLIDS
SOLIDS
DRIVE
CL_2
SEC. SED.
PRIMARY SED. INDUSTRIAL COMPONENT ONLY
DRIVE
DAM
WATER INTAKE HEAD POND
EMERG HOLDING POND
SAN. DISPOSAL SUBSURFACE ABANDON
EXIST. OF. TO P.S.
IND. WASTE
MUNIC. WASTES
H_2O INTAKE
PUMPING STA
MUNICIPAL INTERCEPTOR
BOILER WATER
ST
DRIVE
EXIST OF.
BOTTOM OF BANK
UNOCCUPIED BUILDING
EXIST SCREEN HOUSE
DRIVE
DRIVE
WADE ROAD
TATERSTATE PROCESSING PLANT
DRIVE
BANGOR AND AROOSTOOK RAILROAD
CONDITION A-1-B-2D
100
0
100
FEET

FIGURE 47
PRELIMINARY SITE PLAN
WASHBURN–LOWER SITE

TABLE 70

DESIGN CONDITION - CODING SUMMARY

PRESQUE ISLE SYSTEM

"A" Conditions - Loading alternatives at Potato Service, Inc

A-1 - Operation during Spring of 1971, no in-plant changes
A-2 - In-plant flow reductions and solids recovery
A-3 - Flow reductions, solids recovery, and dry-caustic peel

"B" Conditions - Municipal-Industrial Coordination

B-1 - Potato Service, Inc without municipal wastewaters
B-2 - Potato Service, Inc combined with municipal wastes in following combinations of "A" and "B" conditions:

A-1 + B-2　　A-2 + B-2　　A-3 + B-2

B-3 - Municipal wastewaters alone

major portion of the City. This facility consists of the following units:

Comminutor
Grit Removal
Imhoff Tanks (Covered-Circular)
Sludge Pumps
Heated Digesters
Covered Sand Drying Beds
Effluent Pumps
Chlorination

The original installation relied on the Imhoff tanks for sludge digestion. As loads increased, these digestion facilities proved inadequate and supplemental heated digester units were constructed. Thus the bottom sections of the Imhoff tanks serve no real purpose. During periods of high stream flow the effluent must be pumped from the plant site. While the existing main installation has provided reasonable service in the past, it is inadequate to meet current water quality standards. If this facility is to remain in service extensive modernization and expansion will be required.

The second element of the Presque Isle Facility is the old plant serving the Presque Isle Air Force Base which has been turned over to the City. The former air base properties now serve as an industrial park and other public uses. The old air base plant is an Imhoff tank installation with one circular and one rectangular unit. These units are covered with concrete slabs. Covered sand drying beds served the initial installation.

The old air base plant has received no maintenance for a number of years and is in a state of complete disrepair. Very little actual treatment is currently achieved by this facility, and its value in any expansion program is negligible.

A major expansion is necessary at the Presque Isle plant to meet current water quality standards. The existing facilities are shown on Figure 48 together with the proposed expansion and renovation. The required work to the Presque Isle plant to handle the waste loads presented in the previous section is summarized as follows:

1. Renovate existing headworks structure to provide better heating, ventilation, and backup chlorination facilities.
2. Remove existing glass covered sludge drying beds and construct two aeration tanks to provide 6 1/2 hours aeration.
3. Construct new control building to house influent pumps to lift influent to aeration tanks, control gear, office space, vacuum filters for excess solids dewatering, expanded laboratory and workshop.
4. Abandon existing air base Imhoff tank plant, fill all units, grade and landscape the area.
5. Divert flow now entering old air base plant to the proposed aeration tanks. Provide comminutor and flow measuring device at inlet to aeration units.
6. Convert existing Imhoff tanks to secondary sedimentation tanks. Renovate superstructures and improve ventilation.
7. Construct supplemental secondary sedimentation tank.
8. Renovate sludge pumping station to provide for activated sludge return to the aeration tanks and transfer of excess solids to the digesters.
9. Maintain existing digesters, renovate to provide better utilization of generated gas, or convert to aerobic units.
10. Construct chlorine contact tank for final effluent chlorination.
11. Supplement and renovate existing effluent pumping facilities.
12. Adjust drives, walks and landscaping to meet additional units described above.

If the above program is completed, the facility should provide the treatment required to meet current water quality programs and the growth needs of the City.

Study indicated that if consideration was to be given to combining the Presque Isle domestic wastewaters with those of Potato Service, Inc for treatment, the plant would be located in the vicinity of the Potato Service, Inc processing plant. Thus, studies were made for construction of a facility in the vicinity of the Potato Service, Inc plant to serve either Potato Service, Inc alone, or in combination with the Presque Isle municipal wastewater.

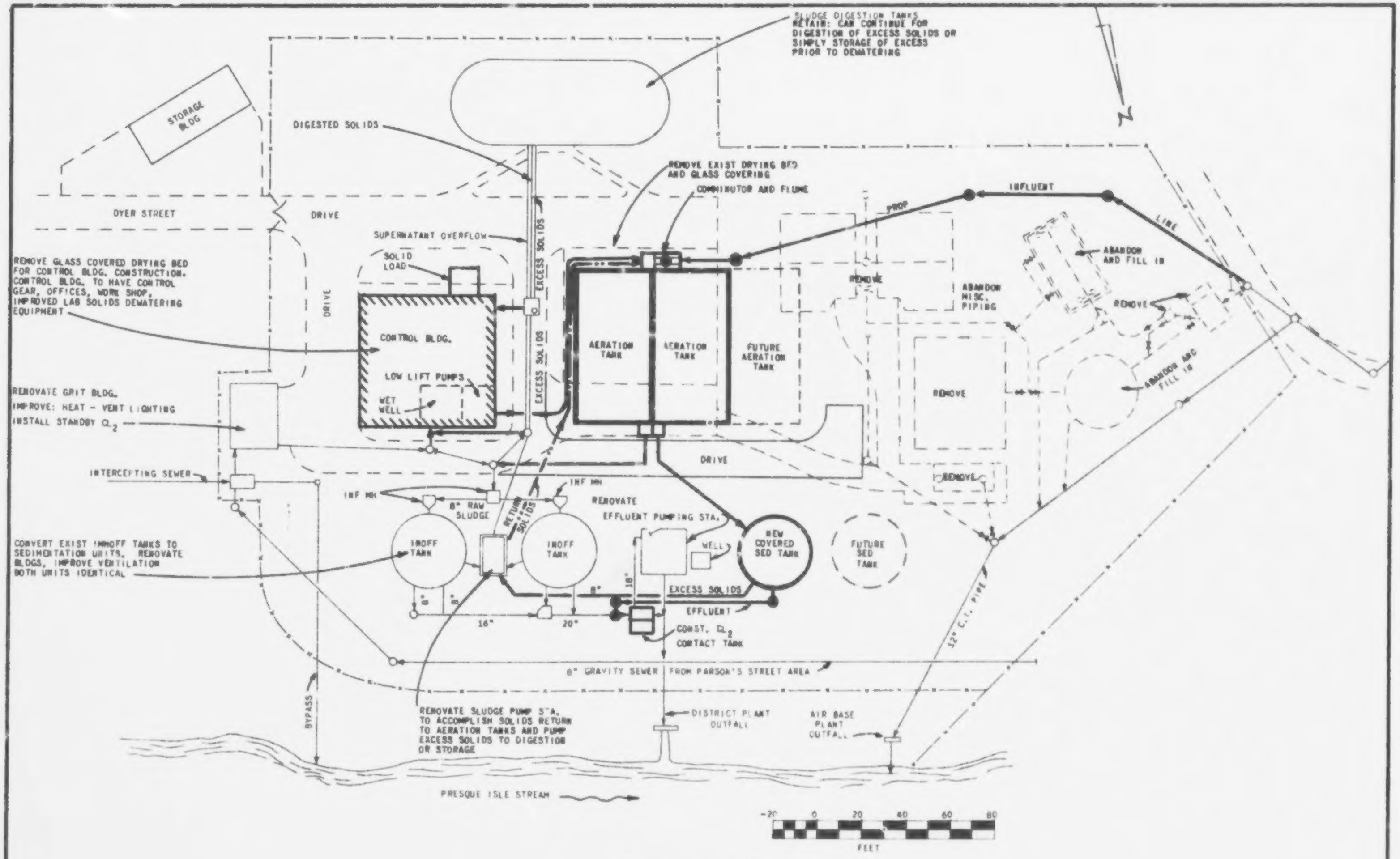

SLUDGE DIGESTION TANKS RETAIN: CAN CONTINUE FOR DIGESTION OF EXCESS SOLIDS OR SIMPLY STORAGE OF EXCESS PRIOR TO DEWATERING
STORAGE BLDG
DIGESTED SOLIDS
N
REMOVE EXIST DRYING BED AND GLASS COVERING
COMMINUTOR AND FLUME
INFLUENT
PROP
LINE
DYER STREET
DRIVE
SUPERNATANT OVERFLOW
EXCESS SOLIDS
SOLID LOAD
REMOVE GLASS COVERED DRYING BED FOR CONTROL BLDG. CONSTRUCTION. CONTROL BLDG. TO HAVE CONTROL GEAR, OFFICES, WORK SHOP, IMPROVED LAB SOLIDS DEWATERING EQUIPMENT
CONTROL BLDG.
LOW LIFT PUMPS
WET WELL
AERATION TANK
AERATION TANK
FUTURE AERATION TANK
REMOVE
ABANDON MISC. PIPING
ABANDON AND FILL IN
REMOVE
ABANDON AND FILL IN
REMOVE
REMOVE
RENOVATE GRIT BLDG.
IMPROVE: HEAT - VENT LIGHTING
INSTALL STANDBY Cl_2
DRIVE
INTERCEPTING SEWER
INF MH
INF MH
8" RAW SLUDGE
RETURN SOLIDS
RENOVATE EFFLUENT PUMPING STA.
WELL
NEW COVERED SED TANK
FUTURE SED TANK
CONVERT EXIST IMHOFF TANKS TO SEDIMENTATION UNITS. RENOVATE BLDGS. IMPROVE VENTILATION BOTH UNITS IDENTICAL
IMOFF TANK
IMOFF TANK
18"
8"
EXCESS SOLIDS
EFFLUENT
16"
20"
CONST. Cl_2 CONTACT TANK
12" C.I. PIPE
8" GRAVITY SEWER FROM PARSON'S STREET AREA
BYPASS
RENOVATE SLUDGE PUMP STA. TO ACCOMPLISH SOLIDS RETURN TO AERATION TANKS AND PUMP EXCESS SOLIDS TO DIGESTION OR STORAGE
DISTRICT PLANT OUTFALL
AIR BASE PLANT OUTFALL
PRESQUE ISLE STREAM
-20 0 20 40 60 80
FEET

FIGURE 48
EXPANSION-PRESQUE ISLE PLANT

The existing primary sedimentation facilities at Potato Service, Inc are relatively new and should continue to provide good service. All evaluations in this report assume these facilities will remain in service. In addition to the primary facilities, Potato Service, Inc maintains a system of three ponds through which the effluent passes before discharge to the river. The first of these ponds is equipped with limited aeration facilities while the remaining two are anerobic. Total detention time in the pond system probably ranges from 6 to 8 days depending on the solids buildup in the ponds. It appears that a 50 to 60% BOD reduction is obtained across the ponds. However, the suspended solids level in the pond effluent was found to be 2 to 3 times higher than that of the influent, during the spring of 1971. Also, the anerobic conditions of the ponds frequently create nuisance odor conditions in the vicinity of the plant.

The efficiency of these ponds could be increased by providing aeration capacity in all ponds to maintain aerobic conditions. However, studies by Dostal[22] indicate that final clarification of the effluent will be required to achieve consistently acceptable BOD reductions. It is also noted that studies reported by Jordan[3] indicate that aerated pond systems are not economic when over 70% BOD removal is required. This is due to the oxygen demands created by the solids generated which will settle in the ponds. While it may be possible to achieve the required 90% BOD removal by adding aeration and final clarification facilities to the existing ponds, it does not appear to offer the consistent reliability required, and in the long term will be equally costly when compared with new relatively short term aeration facilities.

An activated sludge plant to serve the Potato Service, Inc loading conditions, as outlined in the previous section, is shown in Figure 49. It is proposed to reshape a portion of the first existing pond into an aeration basin which would be concrete lined. Preliminary design assumes surface aeration facilities. Secondary clarifiers would be installed in a section of the third pond as shown, followed by a chlorine contact tank. The control building would house solids pumps, excess solids thickening and dewatering facilities, laboratory facilities, controls etc. The effluent would be discharged to the river through a new diffuser system. This is required to distribute the effluent to achieve mixing and avoid concentrations due to the river's tendency to stratify along the shore. It is noted that dual units are shown. While this is recommended, it may be possible to use a single unit system if handling only the industrial wastes.

The remaining sections of the existing ponds would be drained, cleaned and utilized for storage of treated flow if necessary during critical drought conditions, or in event of a system malfunction. The need for possible drought storage of a portion of the flow is discussed in the companion River Basin Planning Report. The plant configuration would be essentially the same for any design loading with only the unit sizing changing. The tentative key unit sizing for the three industrial design conditions as shown in Table 33 are presented in Table 71.

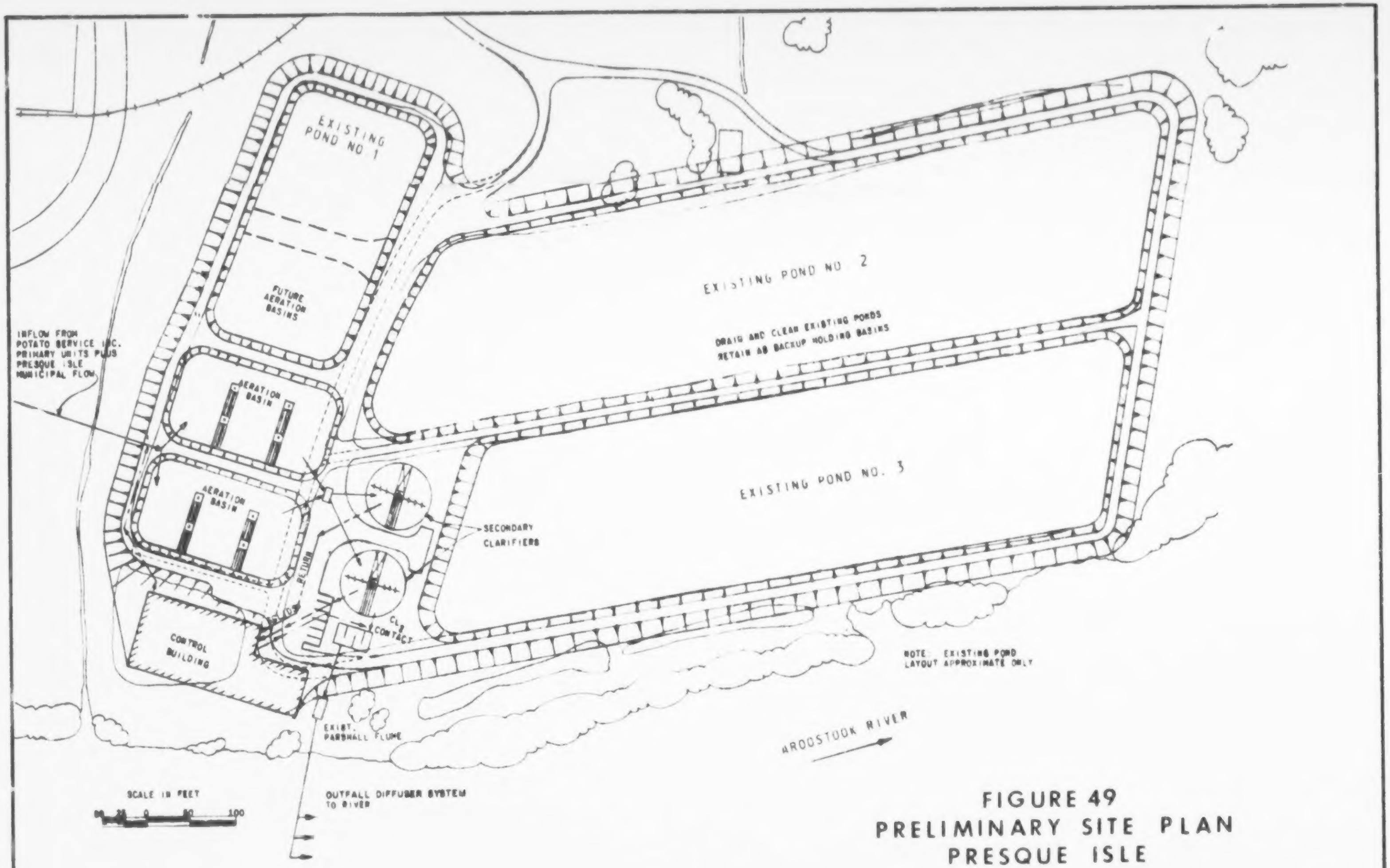

EXISTING POND NO. 1
FUTURE AERATION BASINS
EXISTING POND NO. 2
DRAIN AND CLEAR EXISTING PONDS RETAIN AS BACKUP HOLDING BASINS
EXISTING POND NO. 3
INFLOW FROM POTATO SERVICE INC. PRIMARY UNITS PLUS PRESQUE ISLE MUNICIPAL FLOW
AERATION BASIN
AERATION BASIN
RETURN
SECONDARY CLARIFIERS
CL_2 CONTACT
CONTROL BUILDING
EXIST. PARSHALL FLUME
NOTE: EXISTING POND LAYOUT APPROXIMATE ONLY
AROOSTOOK RIVER
OUTFALL DIFFUSER SYSTEM TO RIVER
SCALE IN FEET
50
25
0
50
100

FIGURE 49
PRELIMINARY SITE PLAN
PRESQUE ISLE

TABLE 71

KEY TREATMENT PLANT SIZING - CONDITION B-1

POTATO SERVICE, INC - POTATO PROCESSING WASTES ALONE - TOTAL UNIT SIZE*

Load Condition at Psi	Aeration Tank Vol CF	Secondary Sed Tank Surf Area	O_2 Req Lbs/hr	Approx Excess Solids Lbs/day	Vac Filter A-Sq Ft	CL_2 Contact Vol CF	CL_2 Req Lbs/day
A-1	270,000	6,710	2,026	28,000	680	5,750	310
A-2	237,000	6,270	1,783	25,000	590	5,375	290
A-3	102,000	4,340	765	10,500	250	3,720	200

*Data for single unit system, divide by two for individual units of Dual System.

NOTE: See Table 70 for design condition coding summary.

The Presque Isle domestic wastewater must be transported to the Potato Service, Inc site if joint treatment is to be considered. The work at the existing Presque Isle plant would be limited to renovations to the existing headworks and expansion of the effluent pumping facilities to lift the flow from the site. New pumps and standard power would be required. The old air base plant would be filled and graded with all flow connected to the pumping facility. The existing Imhoff tanks could be used for partial treatment of excess combined flows if it appears warranted. The existing sand drying beds would be removed, and the digesters would be inactivated.

The wastewater would be carried easterly in an 18 inch force main to the crest line east of US Route 1. From this point gravity flow would be possible, probably paralleling the railroad. It is proposed to carry the interceptor line across the Aroostook River on the existing railroad bridge which is located close to the Potato Service, Inc plant. From the end of the bridge the flow would enter the treatment plant by gravity. The route described above is general in nature, and must be refined during final design to achieve the proper balance between added service area and cost. In any case there appears to be no unusual technical problems in the proposed transfer.

The proposed treatment facility for the combined municipal-industrial wastewaters would be similar to that discussed for the industrial plant, except that dual units would be required. Again the remaining existing pond areas would be utilized for drought storage. Table 72 indicates the key unit sizing of the combined plant for the three assumed load conditions as previously outlined in Table 70.

Review of Tables 71 and 72 reveal that the aeration volumes increase significantly with the introduction of Presque Isle domestic wastewater.

The data in Table 72 reflects a 12 hour minimum aeration time, as established in the design criteria. In the combined system, the domestic flow is a significant component of the total flow. It may be possible during final design to justify a shorter aeration time for the combined flow. However, for preliminary analysis the sizing presented should be adequately conservative. The size of the secondary sedimentation units likewise increase with the larger flow. These values cannot be significantly reduced in final design. The oxygen requirements, solids generation, and other organic design criteria do not increase significantly with the addition of the domestic wastes in combination with the industrial wastes.

It appears technically feasible to combine the Presque Isle domestic wastewater and the Potato Service, Inc process wastewater for treatment in a single plant. Unit sizing of the combined unit are for the most part smaller than the sum of two small systems. However, the combined system must be analyzed from a cost standpoint before decisions are made. Such an analysis is contained in the next section.

CARIBOU AREA - Preliminary designs have been prepared for expansion of treatment facilities in Caribou to meet the demands of current water

TABLE 72

KEY TREATMENT PLANT SIZING - CONDITION B-2

COMBINED POTATO SERVICE, INC AND PRESQUE ISLE DOMESTIC FLOWS*

Load Condition at Psi	Aeration Tank Vol CF	Secondary Sed Tank Surf Area	O_2 Req Lbs/hr	Approx Excess Solids Lbs/day	Vac Filter A-Sq ft	CL_2 Contact Vol CF	CL_2 Req Lbs/day
A-1	391,000	9,748	2,200	29,000	690	8,400	450
A-2	374,000	9,310	1,900	26,000	620	8,000	430
A-3	296,000	7,380	900	12,000	280	6,320	350

*Data for single unit system, divide by two for individual units of Dual System.

NOTE: See Table 70 for design condition coding summary.

quality requirements. The policy of the Caribou Utilities District calls for inclusion of all industrial wastewaters in its municipal system. All preliminary designs, therefore, are based on joint industrial-domestic treatment for all wastewater generated within the existing in-town service area.

The existing primary treatment plant of the District is located adjacent to the Aroostook River just downstream from the junction of Caribou Stream. The plant provides primary sedimentation with chlorination of the effluent prior to discharge to the river. The plant provides anerobic digester facilities and vacuum filtration equipment for primary solids handling. The high percentage of potato solids in the sludge has made normal digestion impossible. This would be expected from review of data by Hinden and Dunstan[26]. With this condition it has been impossible for the District to handle all the solids removed by the primary sedimentation units. Thus a large portion of the incoming wastewater must be bypassed each day. The solids handling problems at the plant must be solved before the existing sedimentation facilities can function effectively.

Two treatment plant site locations have been evaluated. The first option considers on-site expansion of the existing facilities to secondary treatment. The second option provides only primary treatment at the existing site with the clarified waste transferred to Grimes Mill for biological treatment, either with or without the waste flows from a new processing plant, as loading conditions dictate.

Studies have determined that the potato processing component of the waste flows should receive primary treatment. It is assumed that the internal water management program at the Cyr Bros plant will accomplish the equivalent of primary treatment, and additional sedimentation is not required. The Colby Starch plant effluent is not effectively treated by primary sedimentation. The only component of the in-town Caribou flow that actually requires primary treatment is that from American Kitchen Foods. As the primary units at the existing plant are hydraulically overloaded at the present time, they must either be expanded, or the flow must be reduced. Two options have been considered for primary treatment at the Caribou plant.

The alternate designs considered for the Caribou area are coded as presented in Table 73.

Condition B-1, for primary treatment at Caribou, provides for the construction of a new intercepting sewer running southerly from the plant essentially paralleling the existing line. The new interceptor would pick up all flows except those of American Kitchen Foods. In this manner only the American Kitchen Foods wastewater would be carried to the existing primary facilities. If this is accomplished the existing units would be of adequate size to effectively remove the suspended solids component of the American Kitchen Foods waste flow. The waste flows picked up in the new interceptor line would not enter the existing primary units but would pass through new grit removal units and enter the aeration units directly.

TABLE 73

DESIGN CONDITIONS - CODING SUMMARY

CARIBOU - IN-TOWN SYSTEM

"A" Conditions - Loading alternatives for combined American Kitchen Foods, Colby Starch, Cyr Brothers, and municipal wastewaters

A-1 - High industrial load options plus municipal wastewaters
A-2 - Medium industrial load options plus municipal wastewaters
A-3 - Low industrial load options plus municipal wastewaters

"B" Conditions - Primary Treatment Options at In-Town Plant Site

B-1 - Primary - American Kitchen Foods Only
B-2 - Primary - All Flows

"C" Conditions - Grimes Mill Load Option

C-1 - No load from Grimes Mill
C-2 - With load from Grimes Mill

"D" Conditions - Location of Biological Treatment Plant

D-1 - In-town plant site
D-2 - Grimes Mill plant site

Design Combinations for Conditions D-1 and D-2

A-1, B-1, C-1
A-1, B-1, C-2

A-1, B-2, C-1
A-1, B-2, C-2

A-2, B-1, C-1
A-2, B-1, C-2

A-2, B-2, C-1
A-2, B-2, C-2

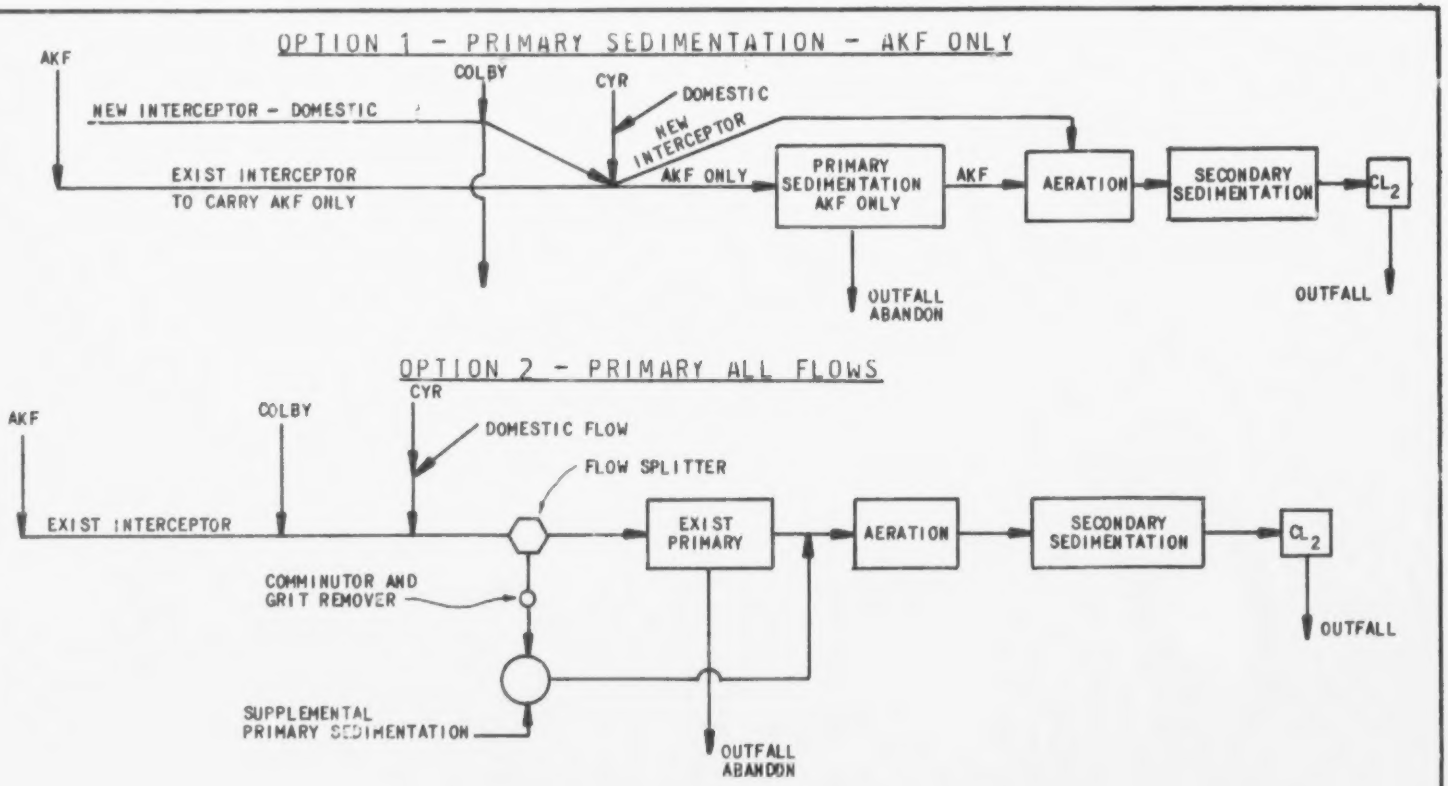
A-3, B-1, C-1
A-3, B-1, C-2

A-3, B-2, C-1
A-3, B-2, C-2

Condition B-2 for primary treatment at Caribou assumes provision of primary treatment for all waste flows arriving at the plant site. To accomplish this within design loading restrictions, supplemental sedimentation facilities will be required. A single supplemental circular unit is proposed. It will also be necessary to install flow apportionment facilities and supplemental grit removal units.

Following either of the primary options outlined above the wastewater flow will enter the biological portion of the system. The design criteria for this portion of the plant will be similar to the other plants previously discussed. Figure 50 shows a schematic system diagram illustrating the two primary treatment options.

Preliminary designs for expansion of the in-town plant have been prepared for the three in-town industrial loads presented previously in Table 35


OPTION 1 - PRIMARY SEDIMENTATION - AKF ONLY

OPTION 2 - PRIMARY ALL FLOWS

AKF
COLBY
CYR
DOMESTIC
NEW INTERCEPTOR - DOMESTIC
NEW INTERCEPTOR
EXIST INTERCEPTOR
TO CARRY AKF ONLY
AKF ONLY
PRIMARY SEDIMENTATION AKF ONLY
AKF
AERATION
SECONDARY SEDIMENTATION
CL_2
OUTFALL ABANDON
OUTFALL
AKF
COLBY
CYR
DOMESTIC FLOW
FLOW SPLITTER
EXIST INTERCEPTOR
EXIST PRIMARY
AERATION
SECONDARY SEDIMENTATION
CL_2
COMMINUTOR AND GRIT REMOVER
SUPPLEMENTAL PRIMARY SEDIMENTATION
OUTFALL ABANDON
OUTFALL

FIGURE 50

PRIMARY TREATMENT OPTIONS

INTOWN CARIBOU

(A-1, A-2, and A-3), and for the two primary treatment options discussed above (B-1 and B-2). A third option considers including a significant processing load from the Grimes Mill industrial park. No load from Grimes Mill is designated C-1. The potential Grimes Mill load, as outlined in the previous section, is designated as C-2.

A tentative site layout of the in-town plant expansion is shown on Figure 51. This figure also illustrates both of the primary treatment conditions outlined above. For Condition B-1 the flows from the new interceptor would enter directly into the wet well for lifting to the aeration tanks. The flow from American Kitchen Foods would pass through the existing primary units prior to entry into the wet well. The existing chlorine contact structure would be reconstructed to serve as the wet well for low lift pumping to the aeration tanks.

With Condition B-2, the incoming flow would enter a new headworks structure which would apportion the flow between the existing primary sedimentation units and a new circular clarifier. This structure would also contain additional grit removal facilities. After primary sedimentation the two components would be reunited in the low lift wet well.

The clarified influent would be lifted to aeration tanks, to be constructed northerly of the existing digesters. Due to site limitations rectangular concrete basins are required. The secondary sedimentation tanks will be of similar construction. The effluent would then be chlorinated and discharged to the river in a new outfall diffuser system. A new control building would be provided to house offices, chlorinator facilities, electrical gear and laboratory.

It will be necessary to expand the solids handling facilities at the existing plant. This will be necessary in order to handle the solids removed from the primary clarifiers, and the excess biological solids. If Condition B-1 of the primary system is utilized, the primary solids will be mainly potato solids which could be utilized as a feed by-product. If Condition B-2 is adopted, the solids will be mixed potato solids and primary sewage solids, which would likely prohibit its use for feed. In either event, it is proposed to expand the solids dewatering filters to handle the primary solids load. Similar to other plans, it is proposed to dewater this material on vacuum filters without digestion. Chemical conditioning facilities will in all probability be required.

The excess biological solids will require thickening prior to vacuum filtration. It is proposed to convert the existing digesters into thickening units, either air flotation or gravity, as final design indicates. The solids dewatering filters will be further expanded to dewater the excess biologic solids.

The remaining work at the in-town plant would be general reconditioning of existing facilities and site adjustments as required for the new plant units. The adjustments and expansions as proposed should provide the treatment necessary to meet current water quality standards.

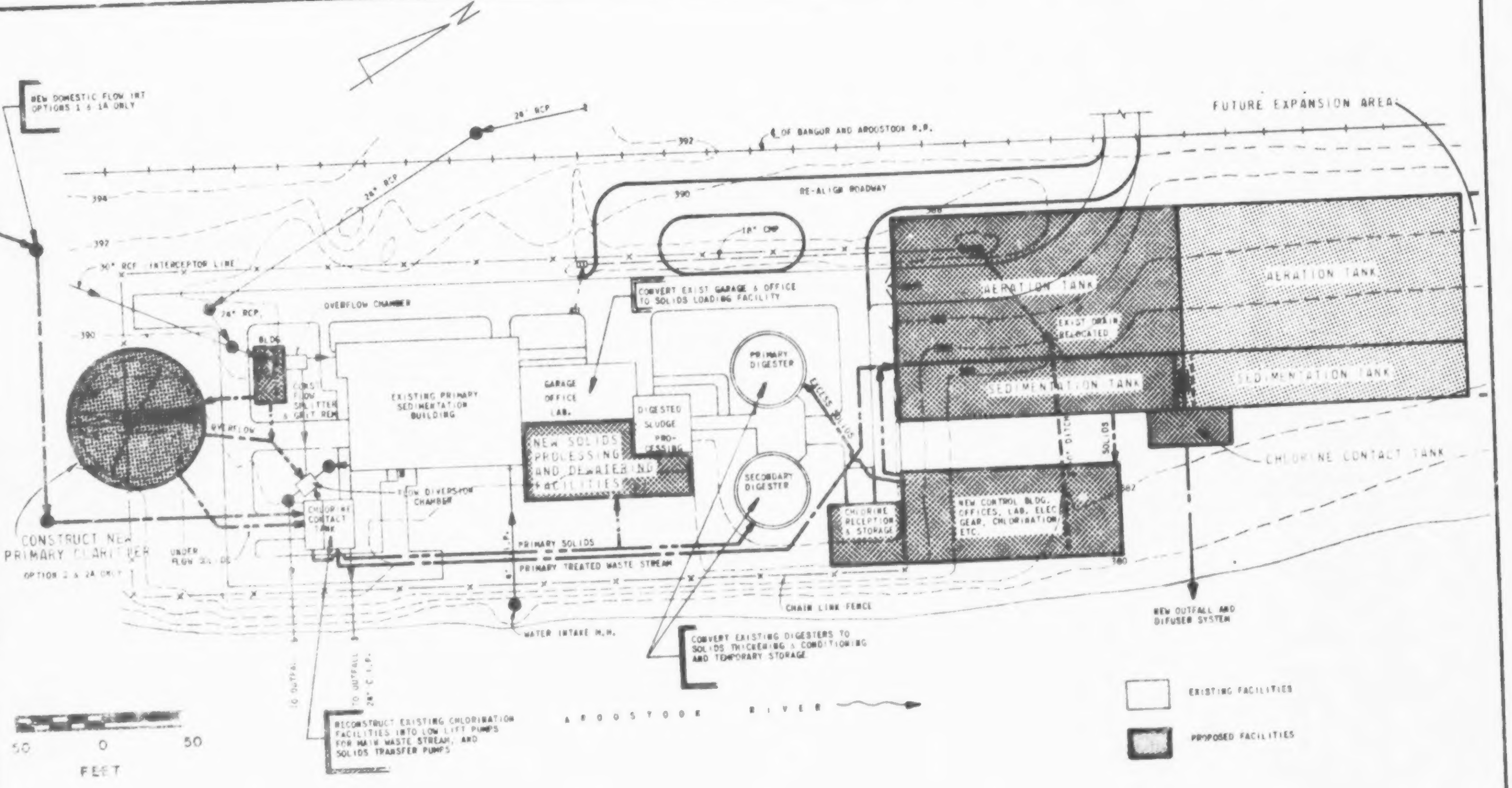

NEW DOMESTIC FLOW INT
OPTIONS 1 & 1A ONLY
24" RCP
392
394
392
390
30" RCP INTERCEPTOR LINE
24" RCP
24" RCP
OVERFLOW CHAMBER
BLDG
CONST FLOW SPLITTER & GRIT REM
EXISTING PRIMARY SEDIMENTATION BUILDING
GARAGE
OFFICE
LAB.
CONVERT EXIST GARAGE & OFFICE TO SOLIDS LOADING FACILITY
DIGESTED SLUDGE PRO-CESSING
NEW SOLIDS PROCESSING AND DEWATERING FACILITIES
OVERFLOW
FLOW DIVERSION CHAMBER
CHLORINE CONTACT TANK
CONSTRUCT NEW PRIMARY CLARIFIER
OPTION 2 & 2A ONLY
UNDER FLOW SOLIDS
PRIMARY SOLIDS
PRIMARY TREATED WASTE STREAM
WATER INTAKE M.H.
TO OUTFALL
TO OUTFALL 24" C.I.P.
RECONSTRUCT EXISTING CHLORINATION FACILITIES INTO LOW LIFT PUMPS FOR MAIN WASTE STREAM, AND SOLIDS TRANSFER PUMPS
50 0 50
FEET
℄ OF BANGOR AND AROOSTOOK R.R.
RE-ALIGN ROADWAY
18" CMP
PRIMARY DIGESTER
SECONDARY DIGESTER
EXCESS SOLIDS
CONVERT EXISTING DIGESTERS TO SOLIDS THICKENING & CONDITIONING AND TEMPORARY STORAGE
AROOSTOOK RIVER
CHAIN LINK FENCE
FUTURE EXPANSION AREA
AERATION TANK
EXIST DRAIN RELOCATED
SEDIMENTATION TANK
AERATION TANK
SEDIMENTATION TANK
DITCH
SOLIDS
CHLORINE CONTACT TANK
CHLORINE RECEPTION & STORAGE
NEW CONTROL BLDG. OFFICES, LAB, ELEC GEAR, CHLORINATION, ETC.
382
380
NEW OUTFALL AND DIFUSER SYSTEM
EXISTING FACILITIES
PROPOSED FACILITIES

FIGURE 51
PRELIMINARY SITE PLAN
CARIBOU–INTOWN

The in-town plant design would remain essentially the same if a waste load from Grimes Mill is included. This flow would either enter the aeration tanks directly or would enter the low lift pumping station depending on grade available. The siting of plant units would then be adjusted to meet flow conditions. Sizing of key plant units is summarized in Table 74.

Under the C-2, D-1 conditions, the flows generated at Grimes Mill would be brought to the in-town plant. These wastes would receive primary treatment at the Grimes Mill processing plant before being introduced into the public system. Topographical conditions require this waste flow be pumped across the river and proceed up stream on the south or east bank. The wastewater sould be lifted to a sufficient elevation to allow gravity flow along the river and recrossing of the river to the plant in an inverted siphon. In this manner some service to the community east of the river in Caribou can be achieved. The general location of this transport link is shown on Figure 52.

As an alternate to the in-town plant expansion described above, preliminary designs were prepared for a biologic plant located in the Grimes Mill area. This is identified as Condition D-2, as presented in Table 73, and includes the same basic options as described earlier.

With the Grimes Mill site alternate, it will still be necessary to provide primary treatment of at least the potato processing component of the waste flows generated in-town. The alternatives for doing this are identical with those of the in-town expansion alternate. It will also be necessary to expand the solids handling facilities at the existing in-town plant. The work required at the existing in-town plant before transporting the wastewater to Grimes Mill is summarized as follows:

1. Expand primary clarification facilities, or construct new interceptor, ie Conditions B-1 or B-2 described in foregoing paragraphs.
2. Expand raw solids conditioning and dewatering capacity by addition of vacuum filter capacity and chemical conditioning facilities. Anerobic digestion would be discontinued.
3. Convert existing chlorine contact chamber to a wet well and pumping facility by structural adjustments and pump installation. Aeration of the wet well would be considered in final design.

The clarified effluent would be pumped across the Aroostook River and lifted to a sufficient elevation to allow gravity flow to the point across the river from Grimes Mill. The wastewater would pass under the river in an inverted siphon system and enter the headworks of a biological treatment plant at Grimes Mill.

By lifting at Caribou the flow would enter the plant at Grimes Mill without additional pumping. This route is essentially the reverse of that shown on Figure 52.

TABLE 74

KEY TREATMENT PLANT SIZING* - CARIBOU

CONDITION D-1 and D-2

Load Condition	New Primary Sed Tank Dia Ft	Aeration Tank Vol CF	Secondary Sed Tank Surf Area	O_2 Req Lbs/hr	Primary Solids Lbs/d	Excess Bio Solids Lbs/d	Vac Filter A-Sq ft	CL_2 Contact Vol CF	CL_2 Req Lbs/d
A-1, B-1, C-1	NA	400,400	10,000	2,180	26,600	29,500	1,300	6,416	462
A-1, B-1, C-2	NA	522,000	13,000	2,920	26,000	39,600	1,550	8,366	602
A-1, B-2, C-1	78	400,400	10,000	2,015	39,400	27,200	1,100	6,416	462
A-1, B-2, C-2	78	522,000	13,000	2,710	39,400	36,500	1,180	8,366	602
A-2, B-1, C-1	NA	260,000	6,500	1,440	10,400	19,400	1,000	4,170	300
A-2, B-1, C-2	NA	380,000	9,480	2,130	10,400	28,800	1,300	7,100	436
A-2, B-2, C-1	50	260,000	6,500	1,325	29,000	14,300	750	4,170	300
A-2, B-2, C-2	50	380,000	9,480	1,980	29,000	26,600	940	7,100	436
A-3, B-1, C-1	NA	236,000	5,860	810	10,400	10,500	850	3,760	270
A-3, B-1, C-2	NA	356,000	8,884	1,488	10,400	19,700	1,200	5,710	410
A-3, B-2, C-1	50	236,000	5,860	797	14,800	10,300	500	3,760	270
A-3, B-2, C-2	50	356,000	8,884	1,470	14,800	19,400	800	5,710	410

*Data given is for total unit - Divide appropriate units by 2 for individual unit sizing
Data assumes any flow from Grimes Mill has been subjected to primary treatment

NOTE: See Table 73 for design condition coding summary

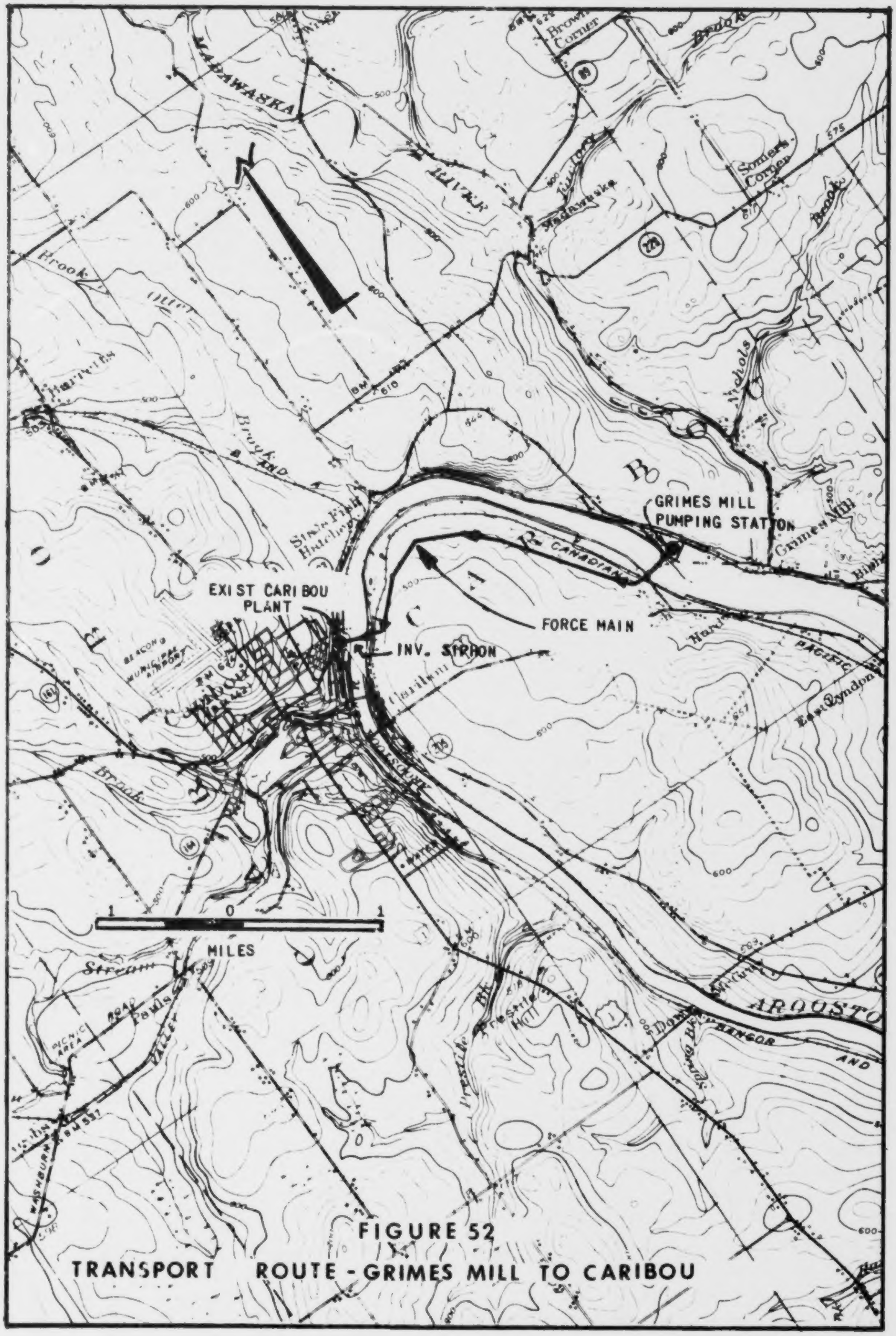

GRIMES MILL
PUMPING STATION
EXIST CARIBOU
PLANT
FORCE MAIN
INV. SIPHON
1
0
1
MILES
MADAWASKA
RIVER
AROOSTOOK
CANADIAN
PACIFIC
BANGOR
AND
Grimes Mill
Somers Corner
Brown Corner
Nichols
Brook
Presque Isle
Caribou
MUNICIPAL AIRPORT
WASHBURN
Stream
PICNIC AREA

FIGURE 52
TRANSPORT ROUTE - GRIMES MILL TO CARIBOU

The wastewater generated at the Grimes Mill industrial park would also enter the plant by gravity. It is assumed that these wastes would receive primary clarification, or equivalent solids removal, before entry into the biological portion of the plant. This could be done at the individual industrial plant, or as a part of the treatment plant headworks.

The biological treatment plant at Grimes Mill would be similar to that proposed at other locations; consisting of lined earthen embankment type aeration basins, circular sedimentation units, chlorination facilities and a diffuser type outfall. Solids handling would consist of thickening and vacuum filtration.

The plant would be located southerly of the Bangor and Aroostook railroad tracks adjacent to the old highway location. A preliminary site plan is shown on Figure 53. The general layout would be similar under all loading conditions, with only the sizing changing. The key unit sizing will be identical with those shown in Table 74 for in-town plant expansion, except that the primary sedimentation tank would be located in-town and the vacuum filter capacity would be divided between the in-town and Grimes Mill location. Only the site configuration and type of construction would change.

It is recommended that additional land easterly of the plant site be purchased for possible future installation of effluent holding ponds for river quality control during drought conditions. The potential need for such facilities is discussed in detail in the companion River Basin Planning Report. It is noted that installation of such a facility is not possible at the in-town location.

Construction of treatment facilities, as proposed at either in-town Caribou or at Grimes Mill, would meet the immediate requirements of water quality standards. The relation of each program to costs and long range water quality objectives is discussed in later sections.

FORT FAIRFIELD AREA - Preliminary designs were prepared for the waste load ranges established for the Fort Fairfield area. Again designs include individual industrial and domestic facilities, and the option of providing improved sedimentation facilities at the A & P plant, or retaining the existing air flotation unit.

Planning to date at Fort Fairfield has been based on construction of separate treatment at two plants, both of which were to be constructed and operated by the Fort Fairfield Utility District. This policy has been based on the District's, and their Consultant's, best judgment of State and Federal requirements for both water quality standards and eligibility for construction grants-in-aid. These requirements, however, have been in a state of flux, and appear to warrant re-evaluation at this time. The studies accomplished under the Northern Maine Regional Treatment Project are, therefore, in no way critical of past planning by the Fort Fairfield Utility District. They are, rather, an attempt to update planning to meet current guidelines and to provide a basis for final decisions by the District.

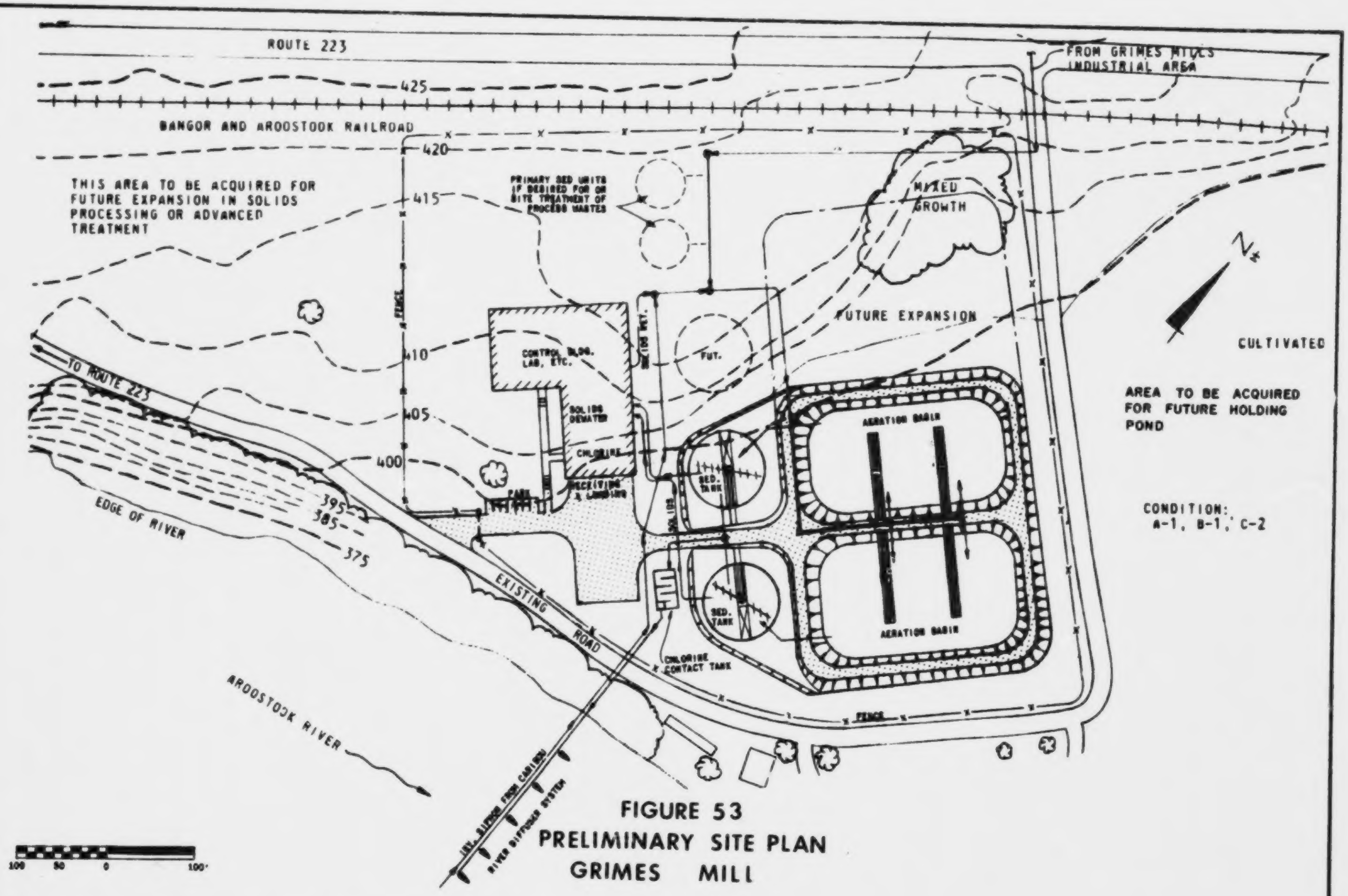

ROUTE 223
FROM GRIMES MILLS INDUSTRIAL AREA
425
BANGOR AND AROOSTOOK RAILROAD
420
415
410
405
400
395
385
375
THIS AREA TO BE ACQUIRED FOR FUTURE EXPANSION IN SOLIDS PROCESSING OR ADVANCED TREATMENT
PRIMARY SED UNITS IF DESIRED FOR ON SITE TREATMENT OF PROCESS WASTES
FENCE
CONTROL BLDG. LAB, ETC.
SOLIDS DEWATER
CHLORINE
RECEIVING & LOADING
PARK
SOLIDS RET.
SOLIDS
FUT.
SED. TANK
SED. TANK
CHLORINE CONTACT TANK
AERATION BASIN
AERATION BASIN
MIXED GROWTH
FUTURE EXPANSION
N
CULTIVATED
AREA TO BE ACQUIRED FOR FUTURE HOLDING POND
CONDITION: A-1, B-1, C-2
TO ROUTE 223
EDGE OF RIVER
EXISTING ROAD
FENCE
AROOSTOOK RIVER
18" SIPHON FROM CARIBOU
RIVER DIFFUSER SYSTEM
100
50
0
100'

FIGURE 53
PRELIMINARY SITE PLAN
GRIMES MILL

The interrelation of the domestic wastewaters of the Town and the processing wastewaters of the A & P plant becomes quite complex when the variable factor of in-plant adjustments at A & P is considered. It is further complicated by the option of improved primary treatment at A & P. Figure 54 illustrates the basic system of wastewater flows at Fort Fairfield with the alternates available. The basic alternate is connection, or non-connection, of the municipal wastes with the industrial wastes. If not connected, two treatment plants will result, or, if connected, a single plant. The A & P plant has the option of inplant changes as discussed in the previous section. It also has the option of retaining its existing air floatation primary treatment facility or replacing it with a gravity sedimentation facility.

The existing air flotation unit is removing about 20 to 25% of the plant BOD, and about 50% of the plant suspended solids. This unit is somewhat less efficient than a well designed gravity sedimentation unit which would be expected to remove 40 to 50% BOD and up to 85% of the suspended solids. In addition, the solids removed from the air flotation unit are very wet and difficult to dispose of in a manner which will not cause additional environmental problems. To assist in evaluation of this option, analysis of treatment facilities have been made, both considering retention of the air flotation unit, and considering its replacement.

The many variables in load conditions at Fort Fairfield create a great number of potential problem solutions. The studied combinations are coded as indicated in Table 75.

TABLE 75

DESIGN CONDITION - CODING SUMMARY

FORT FAIRFIELD SYSTEM

"A" Condition - Loading Alternatives at A & P

A-1 - Operation during Spring of 1971, no in-plant change
A-2 - In-plant flow reductions only
A-3 - In-plant flow reduction, plus dry caustic peel conversion

"B" Condition - Primary Treatment Alternatives

B-1 - Retain existing air flotation unit
B-2 - Build new primary sedimentation tank

"C" Conditions - Municipal-Industrial Coordination

C-1 - A & P without municipal wastes in following combinations of "A" & "B" conditions:

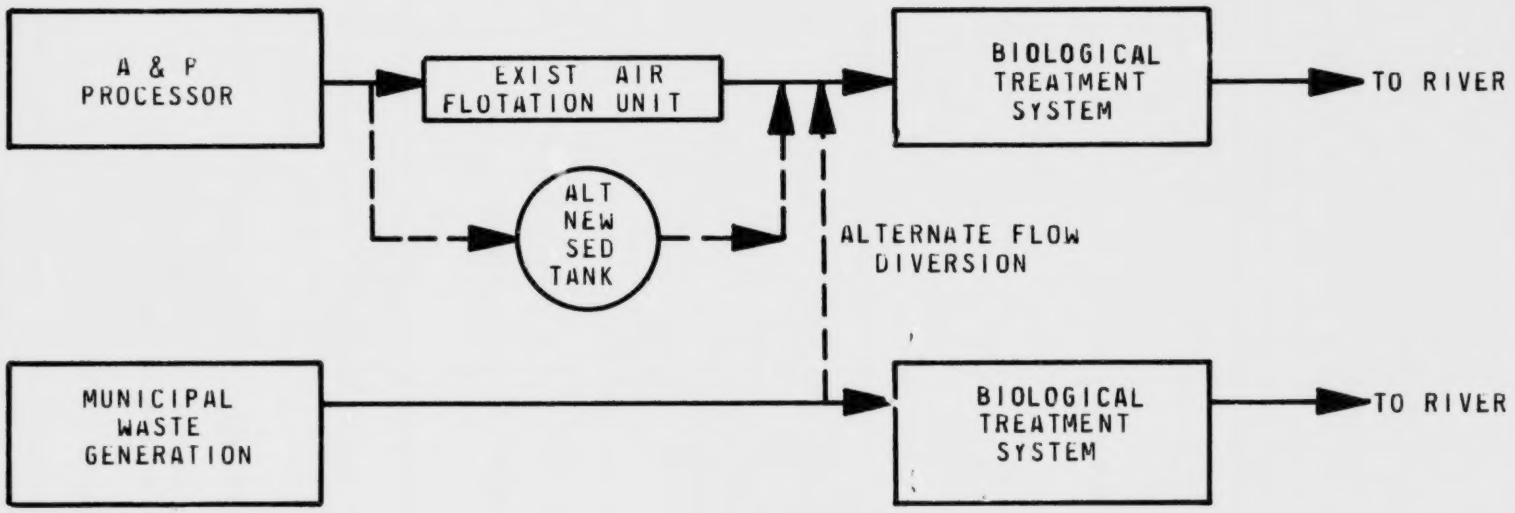

A & P PROCESSOR
EXIST AIR FLOTATION UNIT
ALT NEW SED TANK
BIOLOGICAL TREATMENT SYSTEM
TO RIVER
ALTERNATE FLOW DIVERSION
MUNICIPAL WASTE GENERATION
BIOLOGICAL TREATMENT SYSTEM
TO RIVER

FIGURE 54

DESIGN CONDITIONS - FORT FAIRFIELD - SCHEMATIC

TABLE 75 (CONTINUED)

A-1 + B-1	A-2 + B-1	A-3 + B-1
A-1 + B-2	A-2 + B-2	A-3 + B-2

C-2 - A & P combined with municipal wastes in following combinations of "A" & "B" conditions:

A-1 + B-1	A-2 + B-1	A-3 + B-1
A-1 + B-2	A-2 + B-2	A-3 + B-2

C-3 - Municipal wastewaters alone

The loads to the treatment plant for "A" conditions, reflecting alternative in-plant conditions at A & P, are as previously presented in Table 37. The "B" conditions have been established by assuming the following operational characteristics of the primary treatment units:

UNIT	BOD REMOVED	SS REMOVED
Air Flotation Unit	22%	45%
Gravity Primary Unit	45%	85%

In addition to the outlined load conditions, the preliminary designs reflected either single unit or dual unit construction. The option is similar to that discussed under the Washburn system.

In all cases where a new primary sedimentation tank is to be constructed, a single unit is proposed. This unit will treat only industrial wastewater, and can be taken out of service annually for inspection and repair. The existing air floatation unit will also provide standby potential. In all cases where joint treatment is considered, dual components have been provided. This will be necessary for consideration of governmental grants-in-aid.

Certain key equipment in all configurations, including strictly industrial usage, would be provided in at least two units. These include chlorinators, vacuum filters, aerators, and solids return pumps.

The preliminary design sizing of key treatment plant units is shown on Tables 76, 77, and 78.

TABLE 76

KEY TREATMENT PLANT SIZING - FORT FAIRFIELD

CONDITION C-1 A & P PLANT ALONE - TOTAL UNIT SIZE*

Sub Condition	Primary Sed Tank Dia-ft	Aeration Tank Vol CF	Secondary Sed Tank Surf Area	O_2 Lbs/hr	Excess Solids Lbs/day	Vac Filter A-Sq ft	CL_2 Contact Vol CF	CL_2 Req Lbs/day
A-1 + B-1	None	71,376	1779	556	14,500	220	1143	70
A-2 + B-1	None	70,000	1301	553	14,500	220	837	50
A-3 + B-1	None	34,787	868	228	7,500	135	557	35
A-1 + B-2	42	71,376	1779	392	16,900	225	1143	70
A-2 + B-2	36	52,200	1301	390	16,900	225	837	50
A-3 + B-2	29	34,787	868	161	6,300	105	557	35

*For Dual System - Primary tank as given, subdivide other units as appropriate to obtain sizing of each unit.

NOTE: See Table 75 for design condition coding summary.

TABLE 77

KEY TREATMENT PLANT SIZING - FORT FAIRFIELD

CONDITION C-2 A & P AND DOMESTIC WASTES COMBINED

TOTAL UNIT SIZE*

Sub Condition	Primary Sed Tank Dia-ft	Aeration Tank Vol CF	Secondary Sed Tank Surf Area	O_2 Lbs/hr	Excess Solids Lbs/day	Vac Filter A-Sq ft	CL_2 Contact Vol CF	CL_2 Req Lbs/day
A-1 + B-1	None	101,600	2540	583	14,600	220	1630	100
A-2 + B-1	None	82,600	2060	582	14,600	220	1324	80
A-3 + B-1	None	65,600	1630	230	8,300	100	1046	65
A-1 + B-2	42	101,600	2540	420	17,000	225	1630	100
A-2 + B-2	36	82,600	2060	455	13,000	175	1324	80
A-3 + B-2	29	65,600	1630	190	7,500	100	1046	65

*For Dual System - Primary tank as given, subdivide other units as appropriate to obtain sizing of each unit.

NOTE: See Table 75 for design condition coding summary.

TABLE 78

KEY TREATMENT PLANT SIZING - FORT FAIRFIELD

CONDITION C-3 MUNICIPAL WASTES ALONE - TOTAL UNIT SIZING*

Sub Condition	Primary Sed Tank Dia-ft	Aeration Tank Vol CF	Secondary Sed Tank Surf Area	O_2 Req Lbs/hr	Excess Solids Lbs/day	Vac Filter A-Sq ft	CL_2 Contact Vol CF	CL_2 Req Lbs/day
24 hr Aer	None	93,946	454	30	100	NA	245	25
20 hr Aer	None	78,288	454	30	174	NA	245	25

*For Dual Units - Divide appropriate units by 2.

NOTE: See Table 75 for design condition coding summary.

Review of Tables 76, 77, and 78 suggest the following:

1. If the A & P plant does not install semi-dry caustic peel equipment, and does not install a new sedimentation tank, (Conditions A-1 + B-1, and A-2 + B-2) the increase in general plant size to accommodate the municipal wastes is relatively small.
2. The increased size requirement to handle the municipal wastes becomes greater if A & P installs a new sedimentation tank, and becomes quite significant if dry caustic peel is installed.
3. The aeration volume required for the municipal wastes alone are nearly as great, or greater than, the volumes required for combined waste treatment. This is due to the long aeration time of the municipal extended aeration system. For the municipal plant alone, this is necessary to effectively handle the small volumes of excess solids generated.
4. Based on the data presented above, serious consideration should be given to joint treatment of the domestic and industrial wastewaters.

Preliminary designs indicate that combined treatment has advantages over separate treatment. Cost evaluation of these systems is presented in the next section. It was assumed that the municipal wastewaters would be transported to the site of the A & P air floatation unit for treatment. Earlier studies by the District indicated that, if joint treatment were to be considered, the most economic plant site would be in the vicinity of the A & P plant. This recommendation appears valid as future industrial growth in the community will most likely be in the westerly section of Town.

Preliminary designs were based on the use of concrete tanks for primary and secondary sedimentation, and the chlorine contact tank. The aeration basins were assumed to be of earthen embankment construction with a concrete lining. Fixed platform mechanical aerations were utilized. Air flotation was used for excess activated sludge thickening prior to vacuum filtration. The sedimentation tanks and aeration tanks were left uncovered. Other equipment would be enclosed in the control building. The existing A & P outfall would be utilized. All wastewaters would be pumped to provide hydraulic grade through the plant. Figure 55 shows a typical site diagram for the plant. Other plant configurations not sketched would be similar to those shown except for varying unit sizing.

EASTON AREA - Preliminary designs have been prepared for treatment of wastewaters generated in the Easton area. Designs have considered individual treatment of the industrial and domestic wastewaters in the Town, and combined treatment of the various components.

Under the individual system alternate at Easton, the Town facility was designed for the loading conditions as previously presented in Table 38. Vahlsing, Inc and Maine Sugar Industries plants were designed for each

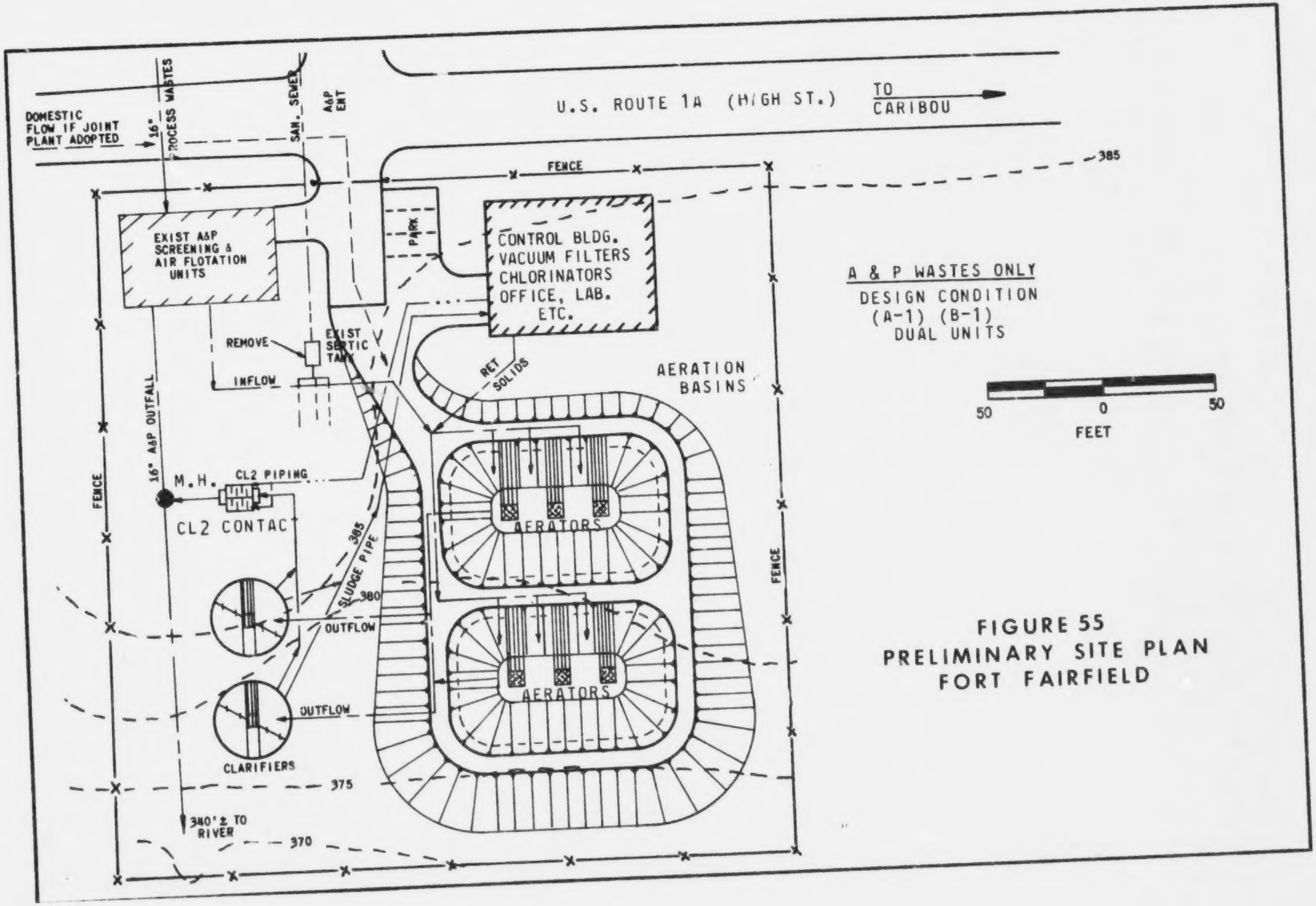

U.S. ROUTE 1A (HIGH ST.)
TO CARIBOU
DOMESTIC FLOW IF JOINT PLANT ADOPTED
16" PROCESS WASTES
SAN. SEWER
A&P ENT
FENCE
385
EXIST A&P SCREENING & AIR FLOTATION UNITS
PARK
CONTROL BLDG. VACUUM FILTERS CHLORINATORS OFFICE, LAB. ETC.
A & P WASTES ONLY
DESIGN CONDITION (A-1) (B-1) DUAL UNITS
REMOVE
EXIST SEPTIC TANK
INFLOW
RET SOLIDS
AERATION BASINS
50
0
50
FEET
16" A&P OUTFALL
M.H.
CL2 PIPING
CL2 CONTAC
AERATORS
AERATORS
385
SLUDGE PIPE
380
OUTFLOW
OUTFLOW
CLARIFIERS
375
340'± TO RIVER
370

FIGURE 55
PRELIMINARY SITE PLAN
FORT FAIRFIELD

of the three loading conditions, as previously presented in Table 39. The municipal plant was designed as an oxidation pond similar to that recommended in the 1970 Camp Dresser and McKee Report.[33] The general layout and location of the municipal Easton system is shown on Figure 56.

The alternate designs considered for the Easton area are coded as presented in Table 79. In addition to the individual plant designs preliminary designs were prepared for various load combinations within the Town.

TABLE 79

DESIGN CONDITIONS - CODING SYSTEM

EASTON AREA

"A" Conditions - Loading Alternatives at Vahlsing, Inc

A-1 - High Loading Condition
A-2 - Medium Loading Condition
A-3 - Low Loading Condition

"B" Conditions - Loading Alternatives at Maine Sugar Industries

B-1 - High Loading Condition
B-2 - Medium Loading Condition
B-3 - Low Loading Condition

"C" Conditions - Municipal-Industrial Coordination

C-1 - Industrial wastes treated jointly
C-2 - Industrial and municipal wastes combined
C-3 - Municipal wastewaters alone

Biological Treatment Combinations Studied

A-1, B-1, C-1	A-1, B-2, C-1	A-1, B-3, C-1
A-2, B-1, C-1	A-2, B-2, C-1	A-2, B-3, C-1
A-3, B-1, C-1	A-3, B-2, C-1	A-3, B-3, C-2
A-1, B-1, C-2	A-1, B-2, C-2	A-1, B-3, C-2
A-2, B-1, C-2	A-2, B-2, C-2	A-2, B-3, C-2
A-3, B-1, C-2	A-3, B-2, C-2	A-3, B-3, C-2

"D" Conditions - Disposal of Easton Biological Treatment Effluent

D-1 - Pump treated effluent to the Aroostook River
D-2 - Land disposal of treated effluent by spray irrigation
D-3 - Advanced waste treatment (99% BOD Removal) for disposal to Prestile

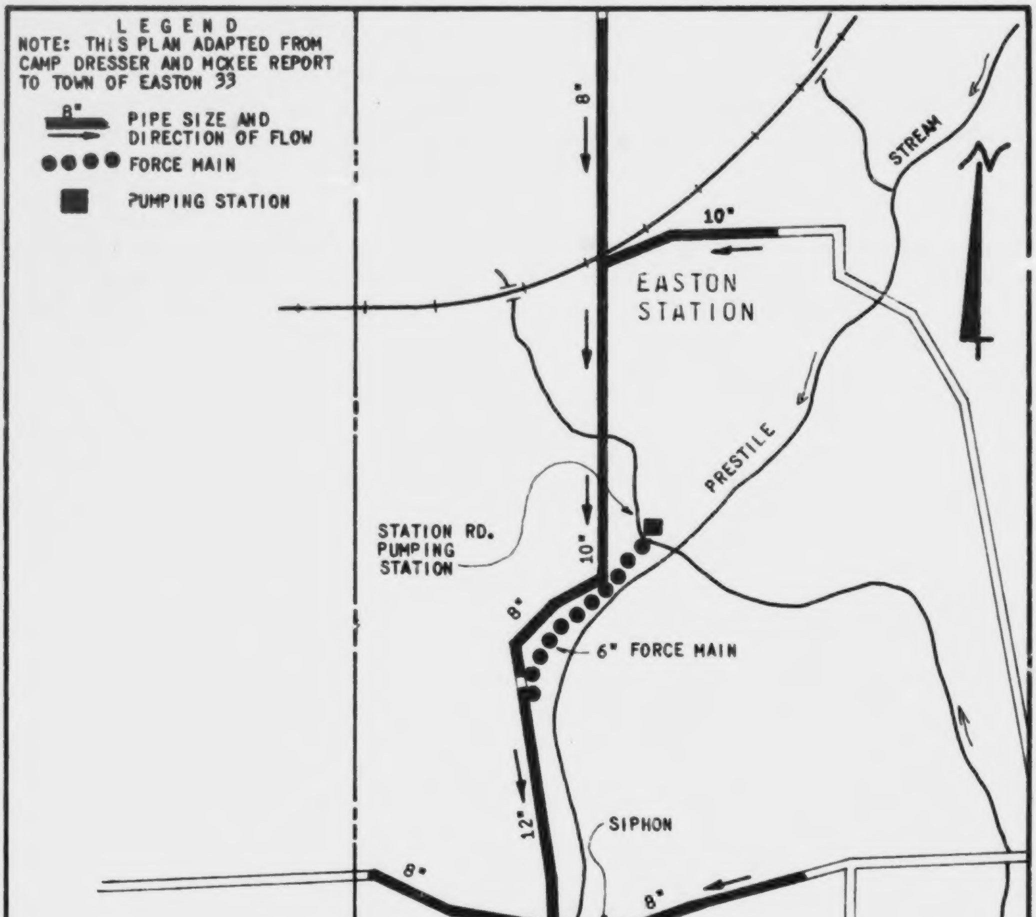

LEGEND
NOTE: THIS PLAN ADAPTED FROM CAMP DRESSER AND MCKEE REPORT TO TOWN OF EASTON 33
8" PIPE SIZE AND DIRECTION OF FLOW
●●●● FORCE MAIN
■ PUMPING STATION
STREAM
10"
8"
EASTON STATION
PRESTILE
STATION RD. PUMPING STATION
10"
8"
6" FORCE MAIN
12"
SIPHON
8"
8"
EASTON
STREAM
8"
PRESTILE
MAIN PUMPING STATION
10" FORCE MAIN
PRESQUE ISLE
EASTON
OUTFALL
FUTURE CELL
STABILIZATION LAGOONS
0
1000
2000
FEET

FIGURE 56
EASTON, MAINE
PROPOSED MUNICIPAL SEWERAGE SYSTEM AND TREATMENT FACILITIES

The preliminary design of the Vahlsing, Inc plant was based on the assumption that the equivalent of primary solids removal would be accomplished by the industry prior to entry into the biological plant. The general type of plant and construction would be similar to those discussed for Presque Isle and Caribou. Pumping to the biological system may be required. The actual siting and configuration of the Vahlsing plant must be accomplished during final design due to limited topographic data. However, no unusual construction problems would be anticipated. The key unit sizing of the plant to serve only Vahlsing, Inc is shown on Table 80.

Preliminary design of a plant to serve only Maine Sugar Industries has been approximated using the same design criteria as developed from the potato processing plants. Little firm design data is available for design of activated sludge systems for beet processing plants. However, it is felt that the design criteria used reflects reasonable plant sizing for preliminary evaluation purposes. The plant again would be similar in configuration and construction to previously discussed plants. The sizing of key plant units for the various loadings at the Maine Sugar Industries treatment plant is shown in Table 81.

Under the C-1 condition the domestic wastes would be treated individually in the oxidation pond system described in the foregoing paragraphs. The combined Vahlsing, Inc - Maine Sugar Industries plant would be located in the vicinity of the adjacent plants at Easton Station. Under the C-2 condition the domestic wastewaters would be pumped to Easton Station for inclusion in the central plant. The combined plant would be similar in configuration to other area plants. The design criteria for the plant units were that assigned to potato processing waste treatment plants even though the beet processing wastes are included. The preliminary sizing of key treatment units is presented in Table 82.

Construction of the combined facilities described above at Easton would be comparable to those proposed in the Aroostook River watershed. However, river studies under the companion River Basin Planning Report indicate that the Prestile Stream cannot accept the effluent from these facilities without violation of the current water quality standards. It would probably be possible to handle only the treated domestic wastewaters through the Prestile system, providing the oxidation ponds had some holding capacity to store flows during low stream flow conditions. Such disposal of the treated industrial wastewaters would not be feasible with the degree of treatment described above.

There are three potential options for ultimate disposal of the treated Easton effluent. These are, 1) pump the treated effluent to the Aroostook River for disposal (Condition D-1), 2) land disposal of the treated effluent by spray irrigation (Condition D-2), and 3) application of advanced waste treatment technique to the Easton wastes to allow disposal to the Prestile (Condition D-3).

Pumping the treated Easton wastewaters to the Aroostook River (Condition D-1) for disposal would maintain the water quality standards of the Prestile Stream. However, such disposal raises several serious problems. The removal

TABLE 80

KEY BIOLOGICAL TREATMENT PLANT SIZING - VAHLSING, INC*

"A" CONDITIONS

Loading Condition	Aeration Tank Vol CF	Secondary Sed Tank Surf Area	O_2 Req Lbs/hr	Excess Solids Lbs/day	Vac Filter A-Sq ft	CL_2 Contact Vol CF	CL_2 Req Lbs/day
A-1	183,000	4,555	1,045	14,500	340	2,930	210
A-2	122,000	3,033	835	11,500	275	1,950	140
A-3	70,000	1,732	541	7,500	180	1,113	80

*For Dual Units - Divide appropriate units by 2.

NOTE: See Table 79 for design condition coding summary

TABLE 81

KEY BIOLOGICAL TREATMENT PLANT SIZING - MAINE SUGAR INDUSTRIES*

"B" CONDITIONS

Loading Condition	Aeration Tank Vol CF	Secondary Sed Tank Surf Area	O_2 Req Lbs/hr	Excess Solids Lbs/day	Vac Filter A-Sq ft	CL_2 Contact Vol CF	CL_2 Req Lbs/day
B-1	294,000	6,068	666	8,400	200	3,900	280
B-2	174,000	4,334	400	5,000	120	2,785	200
B-3	105,000	2,600	210	2,500	60	1,670	120

*For Dual Units - Divide appropriate units by 2.

NOTE: See Table 79 for design condition coding summary

TABLE 82

KEY BIOLOGICAL TREATMENT PLANT SIZING - EASTON COMBINED*

CONDITION C-1 and C-2

Loading Condition	Aeration Tank Vol CF	Secondary Sed Tank Surf Area	O_2 Req Lbs/hr	Excess Solids Lbs/day	Vac Filter A-Sq ft	CL_2 Contact Vol CF	CL_2 Req Lbs/day
A-1,B-1,C-1	426,000	10,620	1,700	22,200	540	6,800	490
A-2,B-1,C-1	366,000	9,104	1,502	19,700	480	5,850	420
A-3,B-1,C-1	314,000	7,800	1,206	16,000	380	5,012	360
A-1,B-2,C-1	356,000	8,892	1,642	21,800	528	5,114	411
A-2,B-2,C-1	296,000	7,370	1,228	16,300	392	4,736	340
A-3,B-2,C-1	243,000	6,065	891	11,000	280	3,900	280
A-1,B-3,C-1	287,000	7,158	1,256	16,400	400	4,600	330
A-2,B-3,C-1	226,000	5,634	1,050	13,600	338	3,620	260
A-3,B-3,C-1	174,000	4,330	754	10,000	240	2,790	200
A-1,B-1,C-2	452,000	11,268	1,726	22,300	544	7,200	520
A-2,B-1,C-2	392,000	9,754	1,526	19,800	481	6,268	445
A-3,B-1,C-2	340,000	8,448	1,234	16,200	388	5,930	390
A-1,B-2,C-2	382,000	9,540	1,671	22,100	534	6,132	440
A-2,B-2,C-2	322,000	8,018	1,252	16,400	394	5,154	370
A-3,B-2,C-2	270,000	6,714	920	11,800	286	4,316	310
A-1,B-3,C-2	314,000	7,806	1,280	17,000	406	5,016	360
A-2,B-3,C-2	252,000	6,284	1,080	14,300	344	4,038	300
A-3,B-3,C-2	200,000	4,990	778	10,200	245	3,200	230

*Data given for total plant - For dual units divide by 2

NOTE: See Table 79 for design condition coding summary

of the industrial waste flows from the Prestile watershed will have a significant impact on downstream flow conditions on the Prestile. This raises questions of law concerning downstream riparian rights. Low flow augmentation would in all likelihood be necessary.

The introduction of the treated industrial wastewaters into the Aroostook River also creates serious legal questions under current Maine law. Such action has been denied by Maine courts in similar cases. It must also be recognized that introduction of the treated Easton wastewaters into the Aroostook will also utilize a portion of the Aroostooks assimilative capacity. Thus, while transfer to the Aroostook may prove the best economic and technical solution, it poses many difficult legal and administrative problems. These problems are fully discussed in the companion River Basin Planning Report.

Land disposal (Condition D-2) of the industrial wastewaters generated in the Easton area was considered in some detail in the 1970 Camp Dresser and McKee Report.[33] The degree of treatment necessary before such land disposal on potato crop land can be questioned. The 1970 Camp Report suggested only holding basins which would remove suspended solids prior to spray irrigation during the growing season. Capacity would have to be provided to store up to 8 months flow for summer irrigation.

It is felt that such raw application to active potato crops should not be undertaken until research has been accomplished in sufficient detail to assure against health hazards. It should be noted that potato processing wastes have a very high coliform bacteria count, although it is mostly, but not entirely, non fecal types. Studies of sugar beet processing wastes by the Federal Water Quality Administration[31] has indicated a significant potential health problem with high bacteria counts, which included salmonella.

It is also possible that potato plant diseases could be transmitted or concentrated by application of essentially a raw waste to active crop land. This area requires additional plant science research. The effect of relatively high loading of the raw wastes on the soil characteristics is generally unknown. While certain nutrient components are perhaps beneficial, other components could be harmful in the long run. Again research is necessary before a full scale land disposal system appears viable.

The above comments apply to a somewhat lesser degree to a secondary, treated disinfected wastewater. However, its long term effect on crop land soils should not be overlooked. The cost of an extensive spray irrigation system on top of full secondary treatment would also be high.

The rate of application of the effluent to the soil must be gaged to assure essentially no runoff. If any significant runoff occurs, the water quality standards of the Prestile will be violated. This rate of application will vary with several factors, such as soil types, slopes, and especially the climatic conditions. Figure 57, as adapted from the Camp Dresser and McKee Report[33], indicates tentative application rates. When

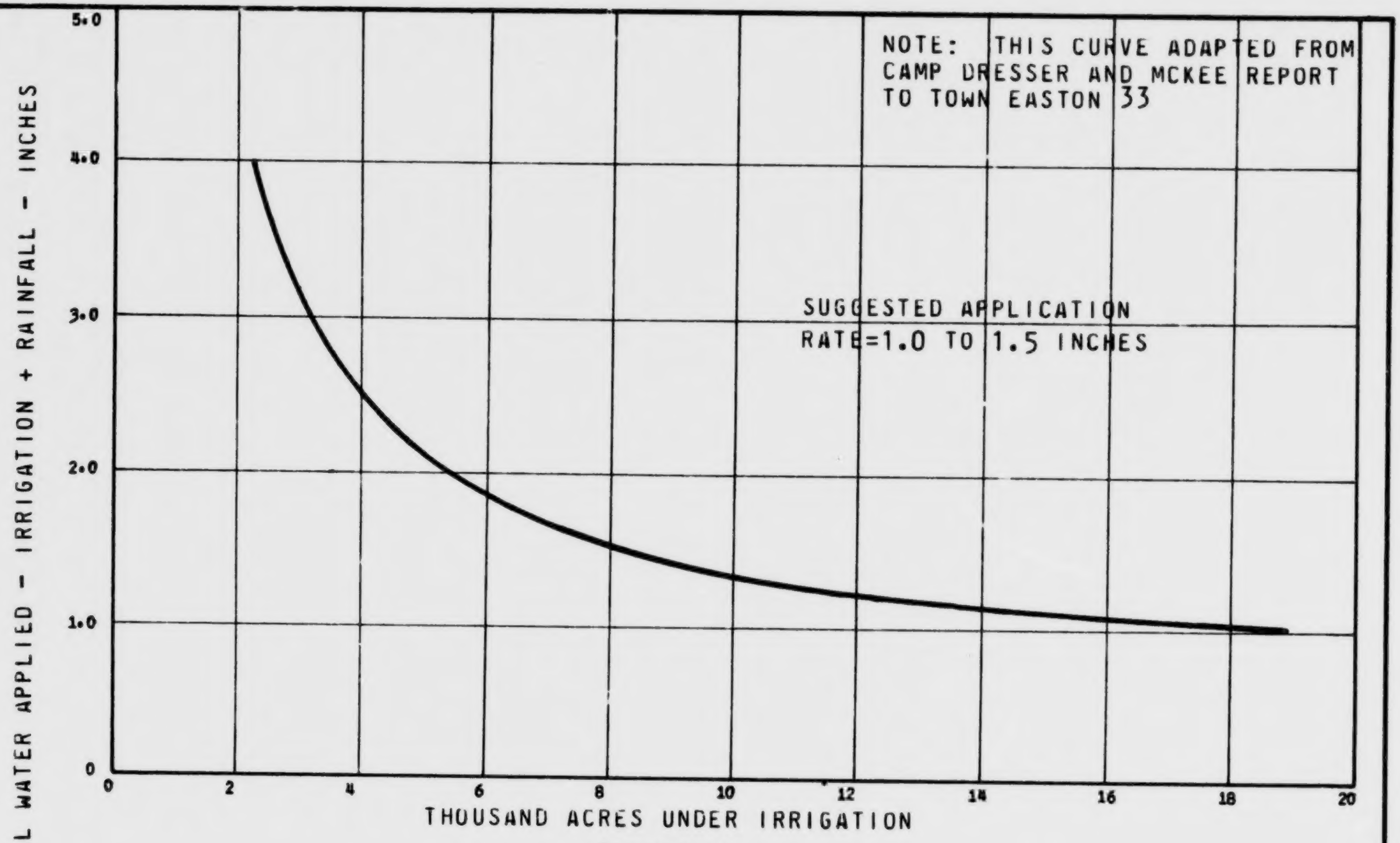


FIGURE 57

IRRIGATION APPLICATION VERSUS ACREAGE REQUIREMENTS design loads are applied to this figure, it is seen that very large land areas would be required. Its availability within a reasonable distance is questioned. It is also noted that during above average wet seasons it will not be possible to dispose of all of the effluent by land irrigation without significant runoff. While it is likely that stream conditions may be higher than normal during such a season, the runoff would undoubtedly violate water quality standards.

A legal question is also posed by a land disposal system. Such a system would constitute a consumptive use of water at Easton and could significantly reduce downstream flow. Again the question of riparian rights can be raised. This, combined with the above described technical problems, make this alternative quite unattractive at this time.

The third alternative (Condition D-3) suggests application of advanced waste treatment techniques to the Easton industrial wastewaters until they are of a quality capable of being placed into the Prestile without violating the classification. Such treatment is technically feasible through precipitation and removal of phosphorous by sedimentation and filtration, plus activated carbon treatment for high organics removal. As the nitrogen content of potato processing wastes is rather low, its removal by ammonia stripping may not be required. BOD removals of 98 to 99% would be required, plus facilities for occasional drought flow storage. This system would not pose the legal problems that the other alternatives do. The economics of this system are discussed in a later section.

From the foregoing discussions it is evident that routine treatment and disposal of the wastewaters generated at Easton is not possible. Complete local advanced waste treatment systems will be required, or consideration must be given to regionalization of the systems.

REGIONAL SYSTEMS

It is generally accepted that the unit costs of wastewater treatment in a larger plant are less than those in multiple smaller plants. This treatment cost saving must be balanced against the added transport cost required to concentrate the wastewater at a single location for treatment. To accomplish the potential economies of scale of larger systems, it is often necessary to regionalize the system. That is, the facilities would be oriented to achieve flow integration uninhibited by political boundaries.

To allow evaluation of the merits of regionalization of the system, preliminary designs have been prepared for numerous possible intercommunity connections. Designs include both the treatment system and the interceptor systems required to transport the wastewaters. In this evaluation the Presque Isle-Easton-Caribou area has been defined as the "Core Area" containing the largest concentrations of population and industry. The Towns of Washburn, Mapleton and Fort Fairfield have been defined as outlying adjacent areas. The analysis first considered the designs required to interconnect the outlying adjacent Towns to the Core Area.

Once the feasibility, or lack thereof, was determined for outlying town interconnection, the potential Core Area system combinations were evaluated.

WASHBURN-PRESQUE ISLE INTERCONNECT - Study of the physical interconnection between Washburn and Presque Isle has been limited to flow from Washburn to Presque Isle. Reversal of this flow to carry the Core Area to Washburn is not considered a viable alternative. Two possible systems were studied as follows:

1. A gravity system for carrying both the domestic and industrial wastewaters to Presque Isle via a gravity interceptor sewer along the river banks.
2. A pressure pumping station system for carrying both the domestic and industrial wastewaters to Presque Isle in a similar manner as the gravity system.

The gravity system provides for carrying both the domestic and industrial wastewaters to the vicinity of US Route 1 in Presque Isle via a gravity interceptor along the river bank. The natural gradient in this section of the river is low, about .006. While this would result in minimal velocities in the interceptor, such a line would be technically feasible. A pumping station would be required in the vicinity of Route 1 in Presque Isle to lift the Washburn flows to the head of the Core Area plant. This Core Area plant, if located in Presque Isle would be in the vicinity of Potato Service, Inc. The pumping station would also serve the local Presque Isle community in the vicinity of the Route 1 and Route 164 intersection. A schematic layout of this transport system is shown on Figures 58A and 58B. It is also noted that Crouseville Village, so called, in Washburn would be served by this interceptor. A minimum of an 18 inch pipe would be required for existing flows, and a 24 inch pipe would be desirable if any significant industrial growth is anticipated at Washburn.

It will be necessary to construct primary sedimentation and solids handling facilities at the Taterstate plant in Washburn prior to entry into the interceptor system. This is necessary because of the minimal grades and flow velocities available in the gravity interceptor. If these solids were carried in the interceptor, it would result in the excessive leaching of organics from the potato solids into a dissolved state. It may also be desirable to chlorinate the waste flow prior to entry into the interceptor system. The chlorine demands would be increased if the potato solids remained. The primary sedimentation units required at Taterstate would be similar to those discussed under the local system options.

Construction of the interceptor along the river will be difficult due to water and the alluvial soil conditions. A heavy allowance for sheeting will be required in any cost analysis. To avoid these conditions an alternate design was considered using a pressure system. A pumping station would be installed in the vicinity of Wade Road in Washburn which would transport the combined domestic-industrial wastewater to the vicinity of Route 1 in Presque Isle in a force main along Route 164. A second

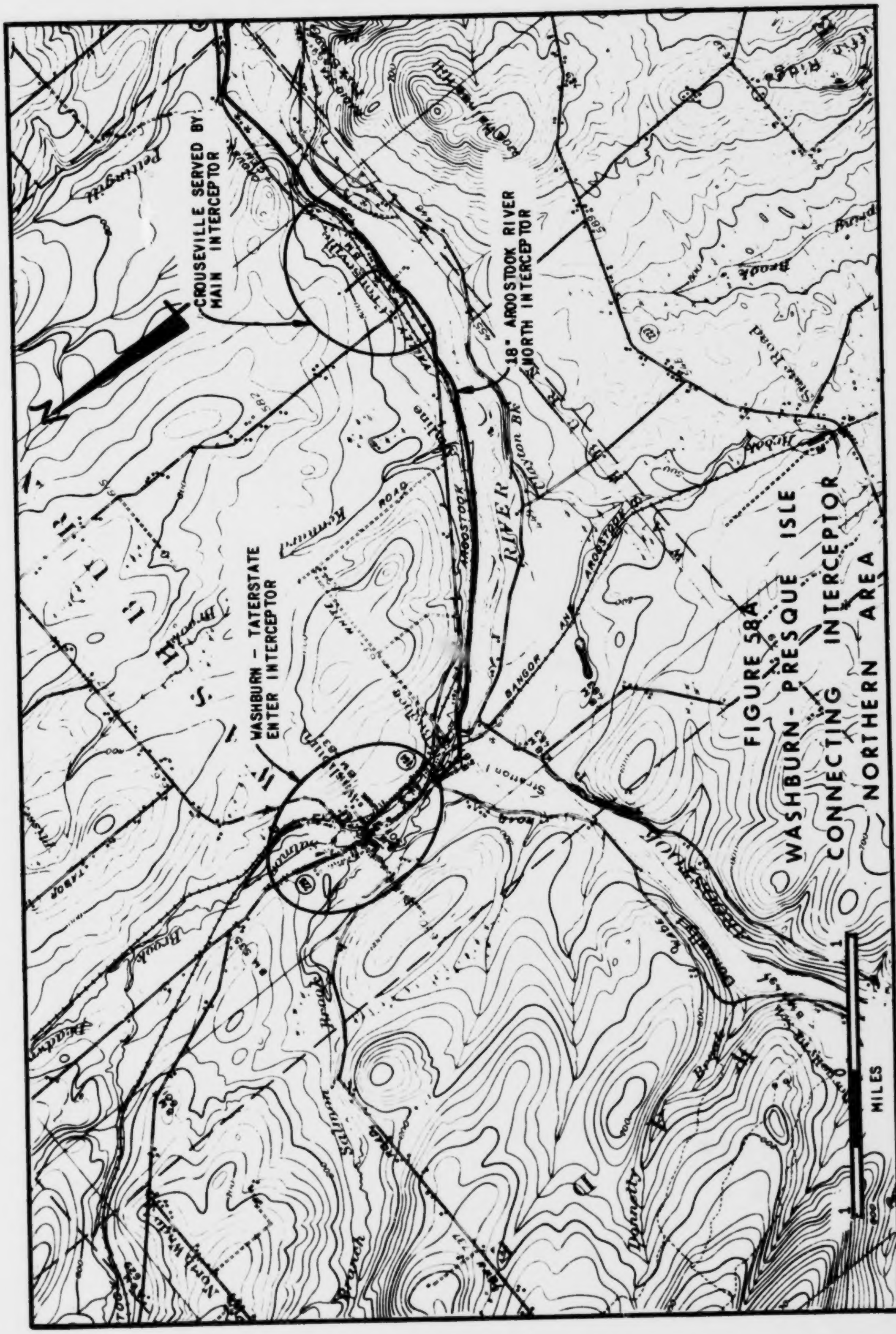

CROUSEVILLE SERVED BY MAIN INTERCEPTOR
18" AROOSTOOK RIVER NORTH INTERCEPTOR
WASHBURN - TATERSTATE ENTER INTERCEPTOR
AROOSTOOK
RIVER
BANGOR AND AROOSTOOK
ROAD
Station 1
MILES

FIGURE 58A
WASHBURN - PRESQUE ISLE
CONNECTING INTERCEPTOR
NORTHERN AREA

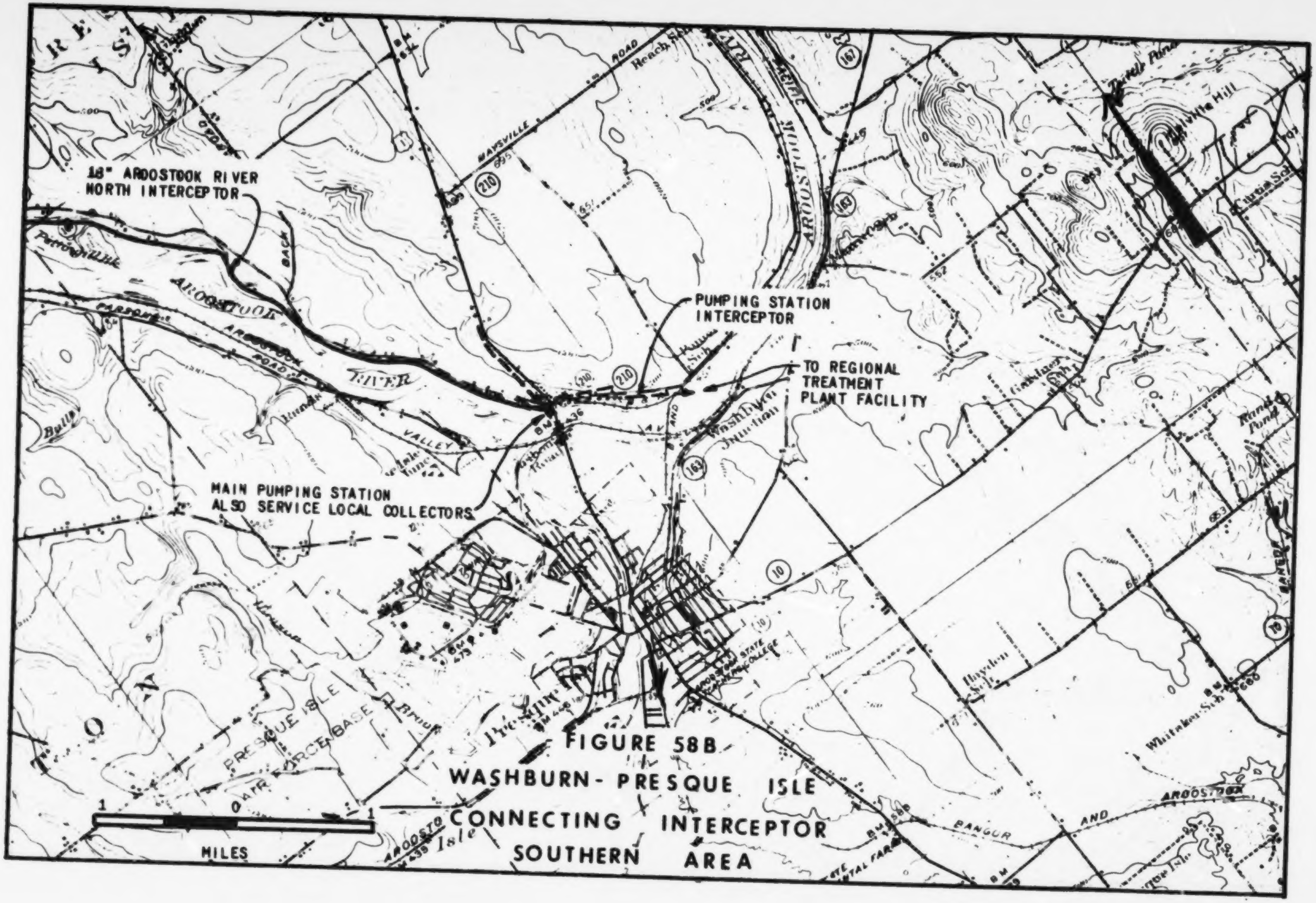

18" AROOSTOOK RIVER
NORTH INTERCEPTOR
PUMPING STATION
INTERCEPTOR
TO REGIONAL
TREATMENT
PLANT FACILITY
MAIN PUMPING STATION
ALSO SERVICE LOCAL COLLECTORS
AROOSTOOK RIVER
PRESQUE ISLE AIR FORCE BASE
1
0
1
MILES

FIGURE 58B
WASHBURN - PRESQUE ISLE
CONNECTING INTERCEPTOR
SOUTHERN AREA

pumping station would be constructed in the vicinity of Route 1 in Presque Isle in a similar manner to the gravity system. Again solids removal at Taterstate would be required prior to entry into the transport system. Chlorination may also be required in this system.

Cost analyses of each of the above interconnect systems are presented in the next section. Based on the data presented in Section VIII, it was concluded that physical interconnection of the Washburn wastewaters with the Core Area is not economically feasible, and Core Area studies should not include these waste flows.

MAPLETON-PRESQUE ISLE INTERCONNECT - An analysis was made of the potential interconnection to carry the Mapleton wastewaters to the Core Area at Presque Isle. Again only a one-way transfer was considered.

Two potential interconnection routes by which the Mapleton wastewaters may be transferred to Presque Isle were studied. These are:

1. Gravity interceptor along Presque Isle Stream to Presque Isle with one pumping station to lift into the Presque Isle Sewer District system.
2. An interceptor system along Route 163 consisting of a series of pumping stations, gravity sewers, and force mains.

These two routes are shown schematically on Figure 59. The system along Route 163 would require a minimum of five pumping stations due to the rolling nature of the topography. The gravity route would pass through extensive areas of swamp land which would increase construction difficulties and costs. As only domestic wastewaters are generated at Mapleton, primary sedimentation prior to entry into the interceptor system would not be required.

An economic evaluation of the Mapleton-Presque Isle interconnection is given in the next section. Based on the data presented therein, it was concluded that physical interconnection of the Mapleton wastewaters with the Core Area is not economically feasible, and Core Area studies should not include these waste flows.

FORT FAIRFIELD-CARIBOU INTERCONNECT - An analysis was made of the potential interconnection between Fort Fairfield and Caribou. Unlike the Washburn and Mapleton analyses, the Fort Fairfield study considered flow in either direction, that is, the Fort Fairfield wastewaters carried to Caribou for treatment, and the Core Area waste flows carried to Fort Fairfield for treatment.

The first analysis assumed that both the domestic and industrial wastewaters from Fort Fairfield would be pumped to Grimes Mill for treatment in the terminal plant of the Core Area regional facility. This would require a major pumping station at Fort Fairfield, in the vicinity of the A & P plant, with a capacity of about 815 gallons per minute. A force

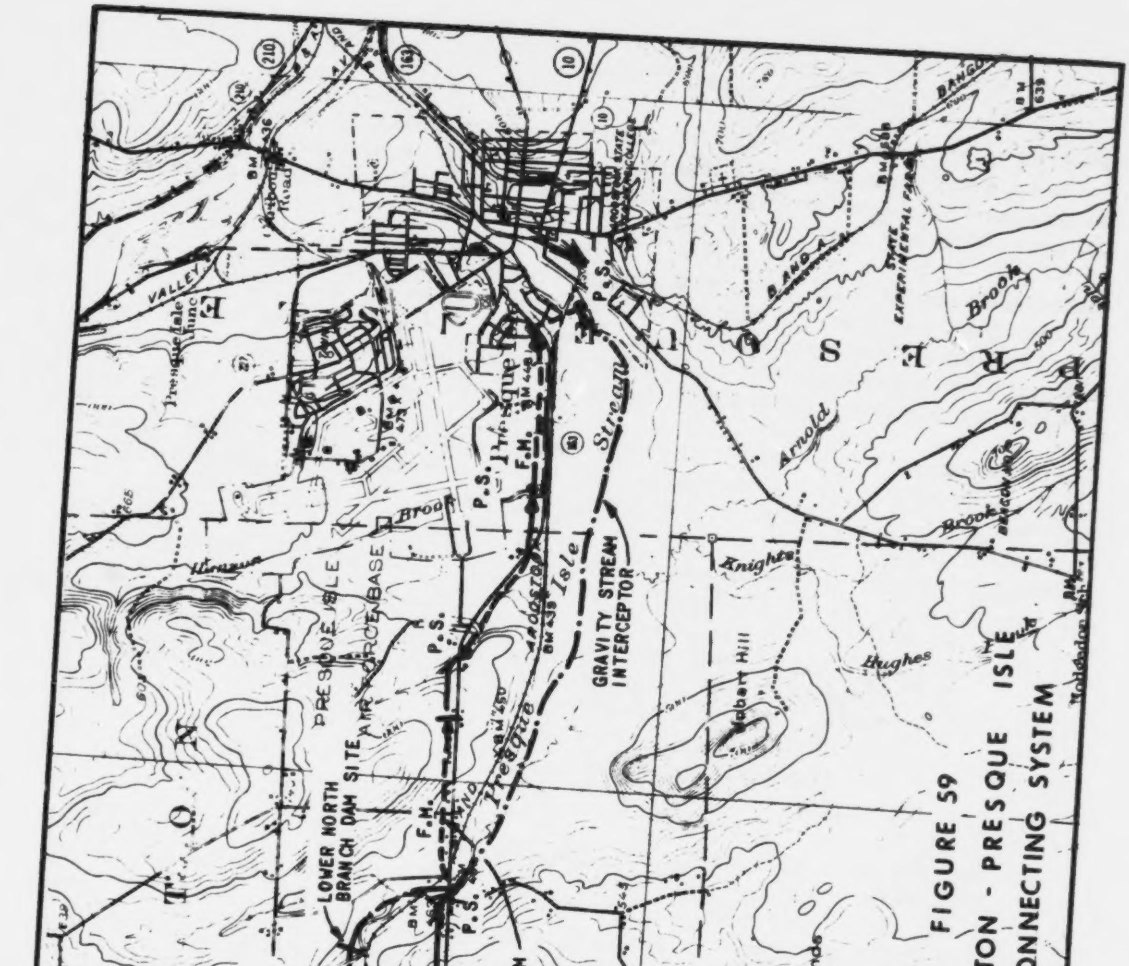

MAPLETON TREATMENT PLANT SITE
GRAVITY INTERCEPTOR
LOWER NORTH BRANCH DAM SITE
PUMPED INTERCEPTOR SYSTEM
GRAVITY STREAM INTERCEPTOR
PRESQUE ISLE AIR FORCE BASE
P.S.
F.M.
RT-163
Mapleton
Presque Isle
Presque Isle Stream
Griffin Ridge
Hobert Hill
Edmunds Hill
Knights
Hughes
Arnold Brook
Chapman
Hillside Sch
BANGOR AND AROOSTOOK
VALLEY
0
1
MILES

FIGURE 59
MAPLETON - PRESQUE ISLE
INTERCONNECTING SYSTEM

main of some 45,000 LF would be required. A minimum 12 inch pipe would be required for existing flows, and a 14 inch pipe would be desirable if any significant industrial growth is anticipated at Fort Fairfield. This pipe would run up the southerly side of the river to a point opposite Grimes Mill where it would cross the river in an inverted siphon. The approximate location is shown in Figure 60.

The piped system will operate satisfactorily while the A & P plant is in operation. However, during periods when the A & P plant is not operating, the domestic wastewaters still must be transferred by the pipeline. With flows less than 50% of design, the transfer time will be increased significantly. This may cause operating difficulties in keeping the flow fresh. It will also be necessary to provide primary sedimentation for solids removal, or its equivalent, at the A & P plant prior to entry into the transport system. The considerations for such primary treatment is similar to those discussed under the Washburn transport system.

A second analysis was made of physical interconnection by carrying the regional wastewaters to Fort Fairfield for treatment. This transfer could be accomplished by either a 48 inch diameter gravity sewer along the northerly side of the river, or by lifting and crossing the river at Grimes Mill, followed by a 36 inch gravity line along the vicinity of the railroad. If gravity flow is utilized to Fort Fairfield, the wastewaters would have to be lifted into the treatment plant. Lifting at Grimes Mill to achieve a better hydraulic grade would probably be the best approach to the design, and has been used for preliminary analysis. This route is essentially the reverse direction of the Fort Fairfield to Grimes Mill route shown on Figure 60. Before carrying the Core Area wastes to Fort Fairfield it is assumed that at least the potato processing component of the Core Area wastewater has received primary treatment for solids removal.

An economic analysis of the Fort Fairfield - Caribou interconnection is presented in the next section. Based on the data presented therein, it was concluded that physical interconnection between Fort Fairfield and Caribou is not economically feasible for either flow direction, and Core Area analysis should not include consideration of the Fort Fairfield waste loads.

CORE AREA INTERCONNECTIONS - Analysis of the outlying Towns, as discussed above, indicates that their interconnection with the Core Area is not economically feasible. Thus the Core Area of Presque Isle, Easton and Caribou should be evaluated on the basis of loads generated within the area. Two basic interconnection options for the Core Area have been evaluated. These alternatives are as follows:

1. Partial Core Area area interconnection combining the waste loads from Presque Isle, including Potato Service Inc, and Easton, including Vahlsing Inc and Maine Sugar Industries. The Caribou wastewaters would be treated as described in the earlier section on individual community systems.

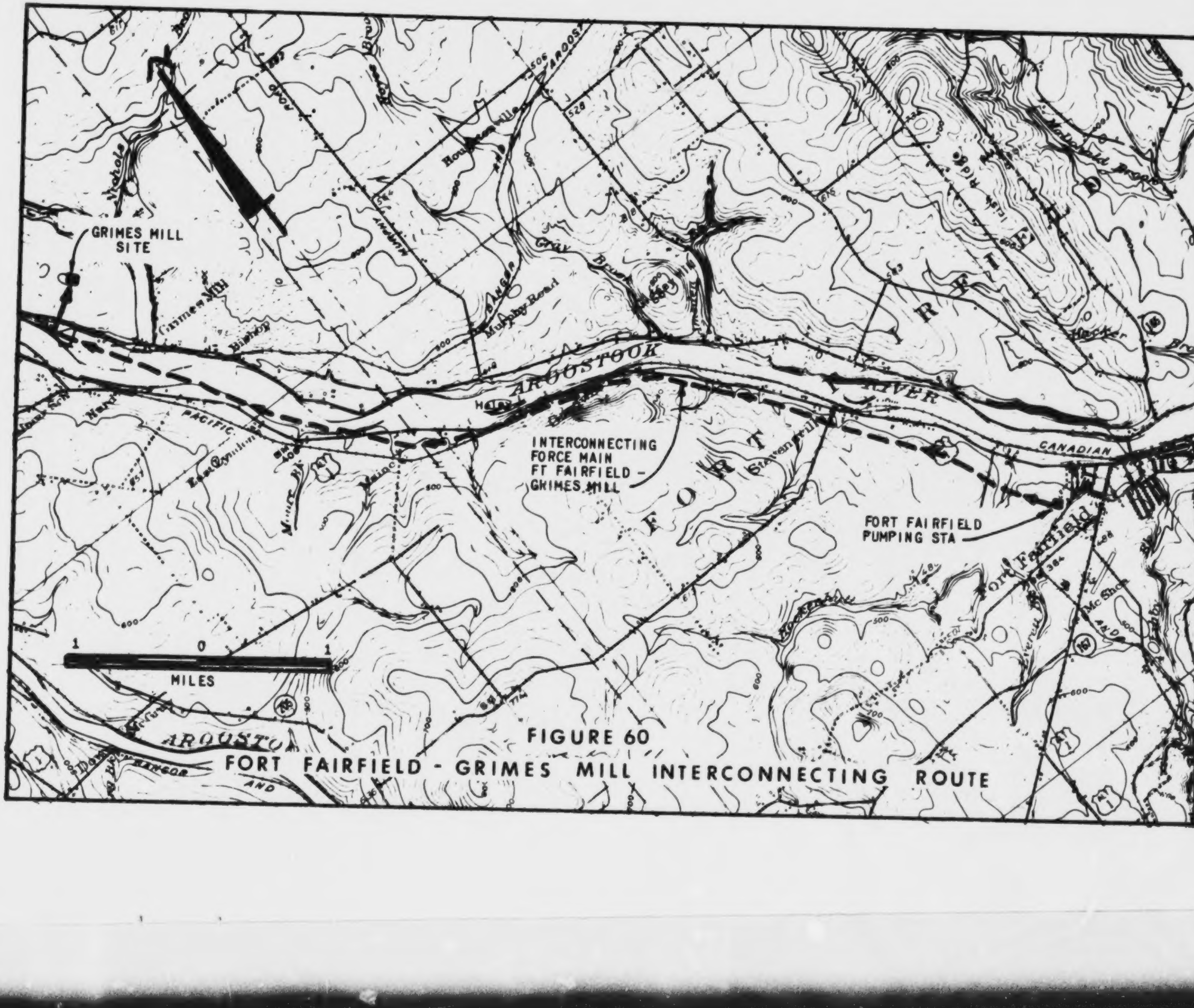

GRIMES MILL SITE
INTERCONNECTING FORCE MAIN FT FAIRFIELD - GRIMES MILL
FORT FAIRFIELD PUMPING STA
AROOSTOOK RIVER
CANADIAN
PACIFIC
FORT FAIRFIELD
Fort Fairfield
MILES
1
0
1

FIGURE 60

FORT FAIRFIELD - GRIMES MILL INTERCONNECTING ROUTE

2. Full Core Area interconnection with all wastes generated in Presque Isle and Easton carried to Caribou for treatment in a single large treatment plant.

Alternate 1 above would basically be an expansion of the plant previously described for the joint Presque Isle-Potato Service, Inc community system to a size capable of serving the various loading which would be contributed from the Easton area. To accomplish the interbasin transfer of Easton flows, the Easton area wastes would be concentrated in the vicinity of the Easton Station industrial complex in a similar manner to the joint domestic-industry option for the Easton community system. A pumping station would be installed at this point to lift the combined Easton wastewaters northwesterly to the watershed divide line. From the divide line the Easton wastes would flow by gravity to the Aroostook River opposite the Potato Service, Inc treatment plant sit. The flow would cross the river and enter the plant headworks via an inverted siphon. It is assumed that the Easton industries will provide the equivalent of primary treatment for solids removal before the wastes are entered into the transport system.

The five contributors to the joint system, ie Presque Isle, Potato Service, Inc, Vahlsing Inc, Maine Sugar Industries, and Easton can contribute almost an infinite number of loading combination to the plant, depending on the various industry loading options as discussed in the previous section. In all analyses it was assumed that all five units would be participating in the system. For preliminary analysis eight typical combination of loadings were studied in detail with preliminary plant designs prepared. The loading conditions for the eight preliminary designs are coded in Table 83.

TABLE 83

LOADING CONDITION CODING SUMMARY

PRESQUE ISLE - EASTON REGIONAL SYSTEM

INDUSTRY LOADING*

Study Condition Designation	Loading Conditions Presque Isle	Loading Conditions Easton
IV-a	A-1 + B-2	A-1 + B-1 + C-2
IV-c	A-1 + B-2	A-3 + B-1 + C-2
IV-g	A-1 + B-2	A-1 + B-3 + C-2
IV-i	A-1 + B-2	A-3 + B-3 + C-2
VI-a	A-3 + B-2	A-1 + B-1 + C-2
VI-c	A-3 + B-2	A-3 + B-1 + C-2
VI-g	A-3 + B-2	A-1 + B-3 + C-2
VI-i	A-3 + B-2	A-3 + B-3 + C-2

*Refer to coding systems established under community systems analysis and design criteria for key to industry loadings.

The Easton-Presque Isle regional plant would be located in the vicinity of the existing Potato Service, Inc ponds, the same location as the joint Presque Isle domestic-Potato Service, Inc industry plant. The type of plant would be similar to that required for Presque Isle and Potato Service, Inc alone, except that a three unit system would be provided rather than the dual unit system proposed for the community system. A preliminary site plan is shown on Figure 61. The configuration would be the same for all plant loadings with only the unit sizes varying. The sizing of the key plant units is given in Table 84.

As discussed under the Easton community systems, the transfer of Easton industrial wastewaters to the Aroostook River watershed raises several legal questions. These are discussed in detail in the companion Basin Planning Report. These studies conclude that low flow augmentation will be required to maintain flows in the Prestile Stream at downstream locations. The most feasible source of such low flows augmentation is the Aroostook River. Preliminary designs call for the installation of a pumping station on the banks of the Aroostook River, adjacent to the inverted siphon which will carry the wastewaters to the treatment plant. This station would be capable of lifting Aroostook River water into the Prestile watershed through a pressure pipe laid in the same trench as the wastewater interceptor.

The low flow augmentation pumping station would tentatively have a capacity of 1000 gpm. The pressure pipe would be 14 or 16 inch diameter. The system would operate only when natural flows in the Prestile below Easton drop to a certain minimum level. The actual value of this level will probably have to be established by legislative action as discussed in the companion Basin Planning Report.

Construction of the Presque Isle-Easton regional system, as described above, would provide a means of meeting the current water quality standards of both the Aroostook River and Prestile Stream. The impact of the added Easton waste load on the river and its capacity to receive treated wastewaters is discussed in detail in the Basin Planning Report.

The second alternative for Core Area regionalization is full interconnection of all wastewaters in the area with treatment at a large regional plant in Caribou. As the Core Area is extensive in size, such interconnection is a major undertaking. Two options have been considered for the Presque Isle-Easton-Caribou interconnection. These are:

1. Standard gravity interceptor pipe system between the Potato Service, Inc location and the existing Caribou treatment plant.
2. Adoption of a treatment-transport system between the Potato Service, Inc location and the existing Caribou plant whereby significant reduction in organic waste load may be accomplished in transit.

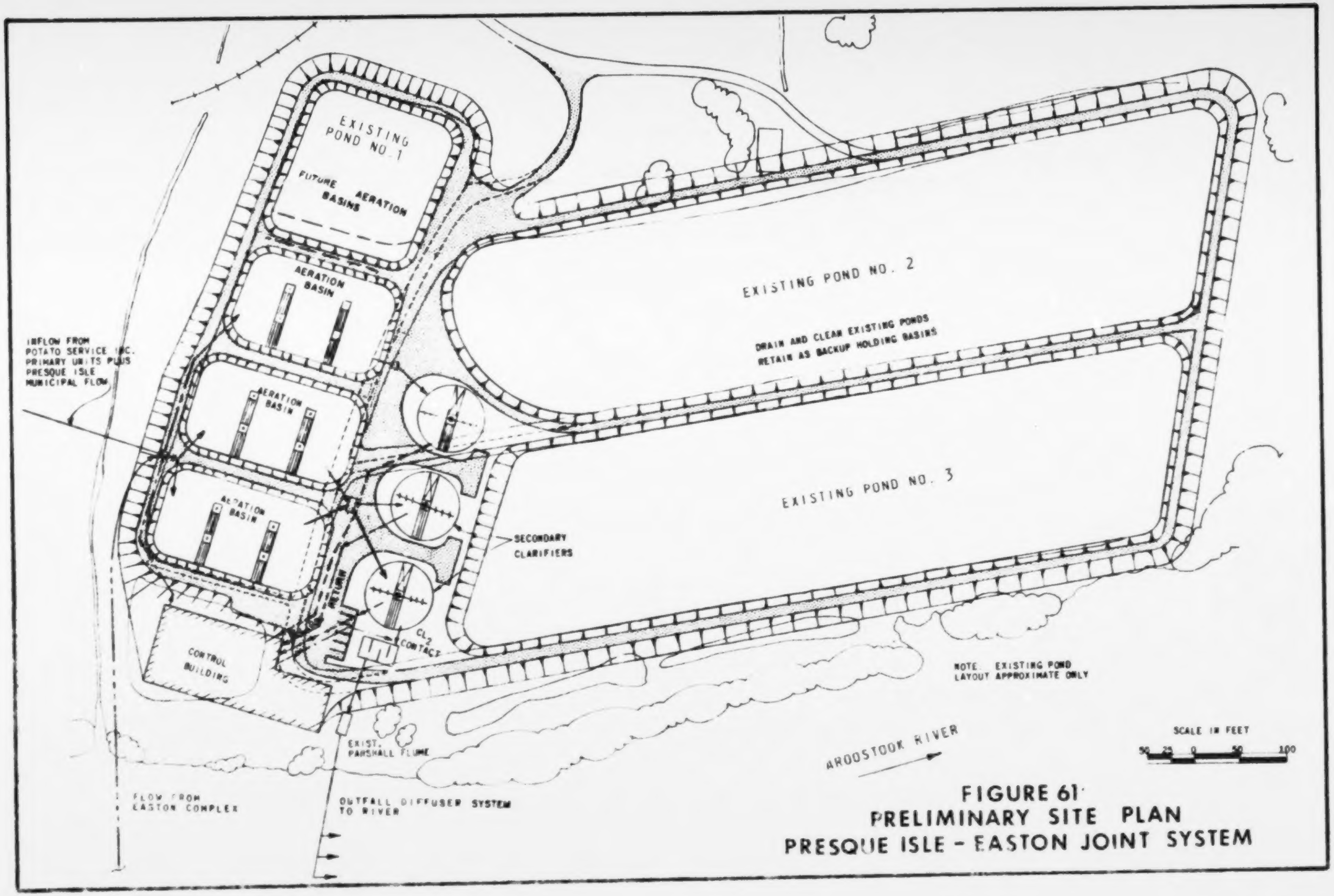

EXISTING POND NO. 1
FUTURE AERATION BASINS
AERATION BASIN
AERATION BASIN
AERATION BASIN
EXISTING POND NO. 2
DRAIN AND CLEAN EXISTING PONDS
RETAIN AS BACKUP HOLDING BASINS
EXISTING POND NO. 3
SECONDARY CLARIFIERS
RETURN
CL_2 CONTACT
CONTROL BUILDING
INFLOW FROM POTATO SERVICE INC. PRIMARY UNITS PLUS PRESQUE ISLE MUNICIPAL FLOW
FLOW FROM EASTON COMPLEX
EXIST. PARSHALL FLUME
OUTFALL DIFFUSER SYSTEM TO RIVER
AROOSTOOK RIVER
NOTE. EXISTING POND LAYOUT APPROXIMATE ONLY
SCALE IN FEET
50
25
0
50
100

FIGURE 61
PRELIMINARY SITE PLAN
PRESQUE ISLE - EASTON JOINT SYSTEM

TABLE 84

KEY TREATMENT PLANT SIZING - PRESQUE ISLE-EASTON REGIONAL PLANT*

Loading Condition	Aeration Tank Vol CF	Secondary Sed Tank Surf Area	O_2 Req Lbs/hr	Excess Solids Lbs/day	Vac Filter A-Sq ft	CL_2 Contact Vol CF	CL_2 Req Lbs/day
IV - a	913,800	22,782	4,152	55,400	1,332	14,743	1,050
IV - c	729,777	18,195	3,336	44,600	1,070	11,694	840
IV - g	704,250	17,559	3,390	45,500	1,092	11,286	810
IV - i	590,880	14,730	2,868	38,500	924	9,468	680
VI - a	747,798	18,645	2,646	34,400	825	11,982	860
VI - c	634,803	15,825	2,130	27,400	660	10,173	729
VI - g	608,898	15,180	2,190	30,400	684	9,756	700
VI - i	495,528	12,354	1,665	21,500	516	7,941	570

*Data given is for total plant sizing - for individual units sizing divide by 3.

NOTE: See Table 83 for design condition coding summary

For evaluation of the extensive Presque Isle-Caribou interconnection system topographic maps were prepared by photogrammetric means at a scale of 1" = 200' with a 5 foot contour interval. Reproducible copies of the photos and contour maps are on file in the Commission office for use in later studies. A soils investigation program was also initated along the interconnect route. The soils report and boring logs are included in this report as Appendix A.

The interceptor route was selected through evaluation of topographic plans, aerial photos and on-site reconnaissance. A high level route generally paralleling the Bangor and Aroostook railroad grade was selected over a low level route along the river. This route is desirable to avoid difficult soils and water conditions along the river. As both the Presque Isle municipal and all Easton wastewater must be pumped to reach the Potato Service, Inc site, and the Potato Service, Inc wastes leave the plant at a relatively high elevation, it is possible to adopt the higher route without additional pumping. The general routing of the interconnection is shown in Figures 62 and 63.

It is noted that the interbasin transfer route for the Easton area wastewaters would shift downstream slightly if the Presque Isle-Caribou interconnect is adopted. This route will shorten the length of pipe required for transfer. This alternative route is also shown in Figure 62.

The standard gravity interceptor would consist of 36 inch reinforced concrete pipe from Potato Service, Inc to the junction with the Easton inverted siphon. Due to grade restrictions and standard pipe sizing, this size would be the same for all design flow combinations at Potato Service, Inc. From the junction with the Easton siphon, the pipe size would increase to 42 inch and extend downstream about two miles where a change in grade will require a further increase to 48 inch, which would continue to the Caribou treatment plant. It may be possible to reduce the 42 inch and 48 inch pipe by one size for the lower flow conditions for the Presque Isle-Easton area. Access manholes would be provided along the line at about 500 foot intervals. There are a number of brooks and streams which must be crossed by pipe bridges, the largest of which will be at Caribou Stream in Caribou.

The piped interconnection as described above is technically feasible. However several special conditions should be considered. The low grades on the line will maintain reasonable velocities (2.5 fps±) during normal flow conditions with the processing plants in operation. With all processing wastes receiving primary sedimentation prior to entry into the system no sedimentation problem in the pipe line would be anticipated. During the summer when only the Presque Isle and Easton municipal flows enter the system the velocities will drop significantly and may be in the range of 0.7 fps. These flows may result in some solids sedimentation in the pipe during the summer. While this could contribute to a potential odor problem, it should not prove an obstacle to the proposed system.

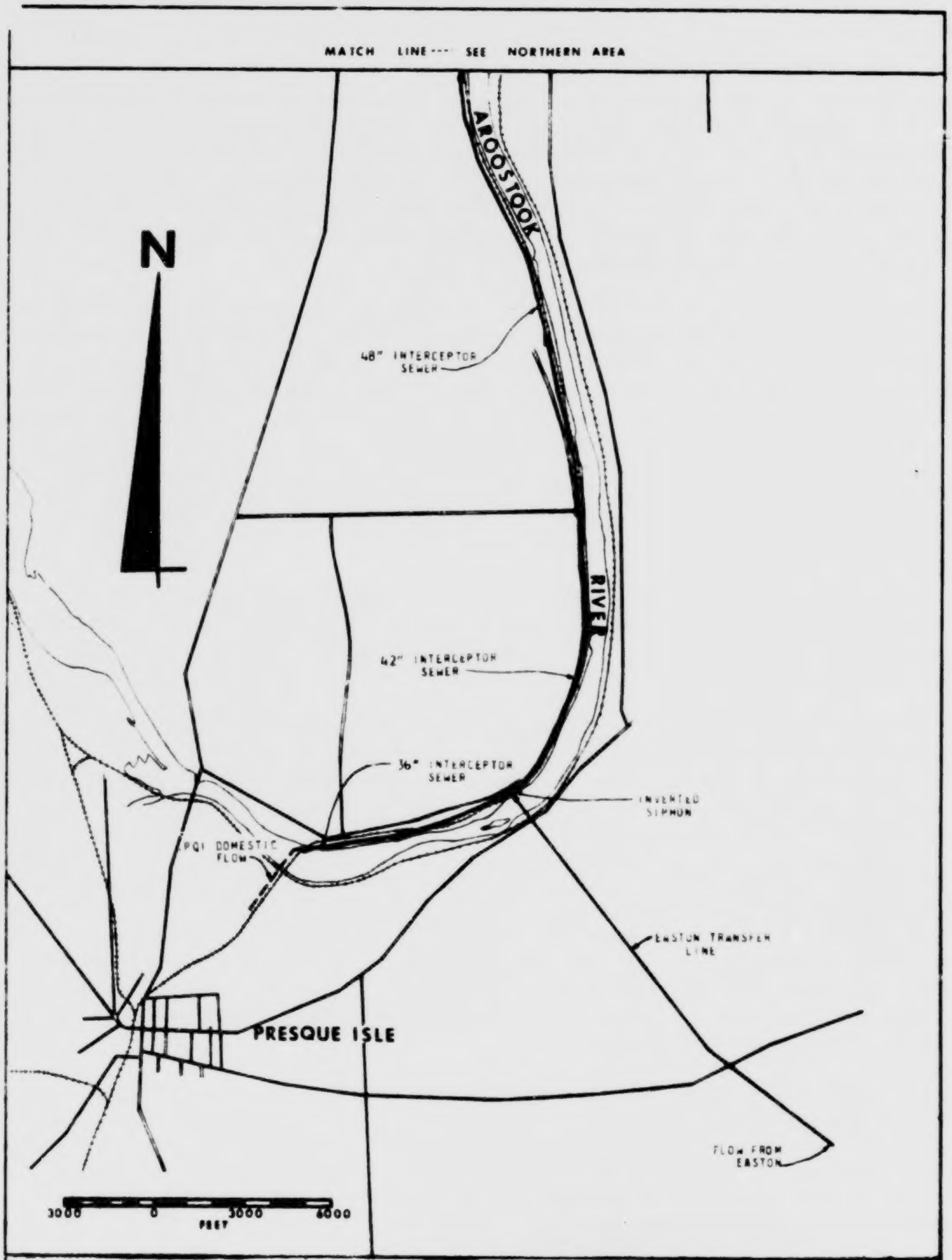

MATCH LINE --- SEE NORTHERN AREA
N
AROOSTOOK RIVER
48" INTERCEPTOR SEWER
42" INTERCEPTOR SEWER
36" INTERCEPTOR SEWER
INVERTED SIPHON
PQI DOMESTIC FLOW
EASTON TRANSFER LINE
PRESQUE ISLE
FLOW FROM EASTON
3000 0 3000 6000
FEET

FIGURE 62
PRESQUE ISLE-CARIBOU INTERCONNECTING ROUTE
SOUTHERN AREA

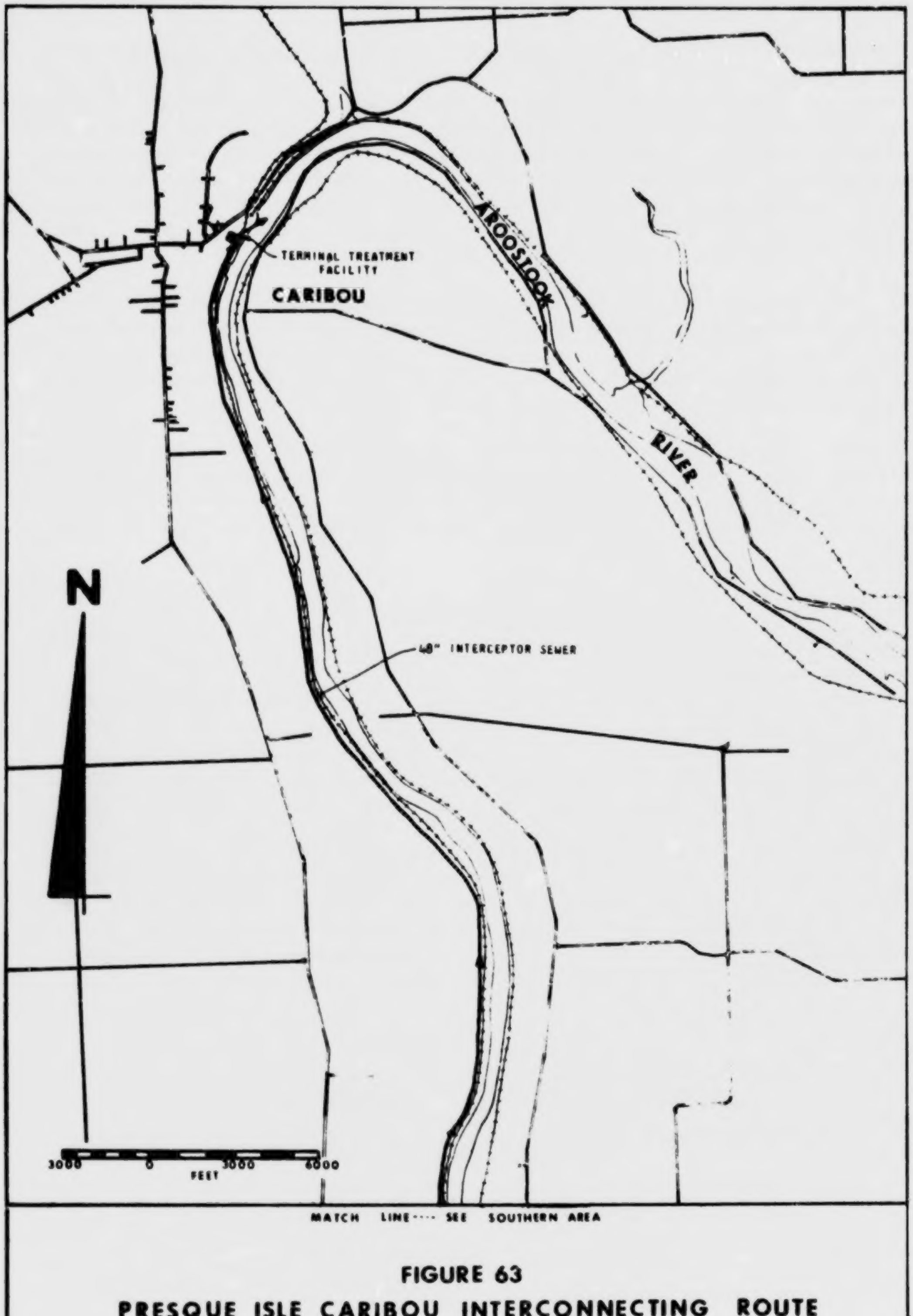

TERMINAL TREATMENT FACILITY
CARIBOU
AROOSTOOK
RIVER
48" INTERCEPTOR SEWER
N
3000 0 3000 6000
FEET
MATCH LINE---- SEE SOUTHERN AREA

FIGURE 63
PRESQUE ISLE CARIBOU INTERCONNECTING ROUTE
NORTHERN AREA

The Presque Isle-Easton flows will be in the pipeline for a period of 8 to 10 hours. This period will allow bacterial action to begin in the wastewater. This bacterial action quickly consumes any residual dissolved oxygen and will lower the pH of the wastewater, as is typical with potato processing wastes. This condition will have two potentially detrimental consequences. The anerobic condition of the wastewaters may well give rise to an odor problem near the Caribou end of the pipeline, and the lowering of pH and possible hydrogen sulfide generation may have a rather corrosive effect on the interior of the pipe.

Design of the piped system will require a protective coating on the pipe interior, and possibly lime addition to counter pH drop and retard bacterial action. Chlorination at intervals along the line could also be considered for controlling bacterial action. Additional studies of anerobic action of potato processing wastes may be warranted.

It should also be noted that anerobic action, if allowed to occur within the pipeline, would provide a certain degree of BOD removal in transit. However, it is probable that such action will be purposely inhibited and the treatment facilities at Caribou should be designed for the full entry BOD load.

Upon reaching the existing Caribou plant, the incoming Presque Isle-Easton wastes must be treated in some combination with the Caribou waste flows. With the 5 input units at Presque Isle and Easton combined with the 5 input units at Caribou an almost infinite range of loadings is possible. For analysis of treatment of the combined Core Area waste loads at Caribou, we have assumed three loading conditions. The high loading condition represents essentially the existing industrial input (as measured in the spring of 1971) plus a twenty year design flow from the domestic component. The low loading condition is the minimum expected and represents maximum in-plant waste reductions by industry, and the omission of the Easton waste load component. This omission would only occur if the current legal restrictions on interbasin transfers prohibit such interconnection. The third loading condition represents a medium condition which could reasonably be reached by various combinations of inplant conservation measures by industry. The design loadings assumed for these three conditions are presented in Table 85.

TABLE 85

REGIONAL TREATMENT PLANT - CARIBOU AREA

LOADING CONDITIONS

Unit	High	Medium	Low
Flow MGD	15.2	9.2	6.1
BOD Lbs/day*	147,330	93,420	40,750

*After Primary Sedimentation

In theory this treatment could be accomplished by expansion of the existing Caribou treatment plant, in a similar manner to that described under the Caribou community system. While this may be possible for the lower range of loadings, the site is not adequate for the medium to higher flow ranges. This fact combined with the potential need for treated effluent storages during drought conditions dictates that if the Presque Isle-Easton wastewaters are to be transported to Caribou, the wastes should be transferred to Grimes Mill for biological treatment.

The incoming Caribou wastewaters would receive primary treatment for solids removal in a similar manner to that described for the Caribou community system. Primary solids would be dewatered at the in-town site. The primary effluent from Caribou would be combined with the incoming Presque Isle-Easton flows in a new wet well structure. Aeration would be provided in the wet well prior to pumping in order to freshen the wastewater. The combined wastes would then be pumped across the river and lifted to achieve gravity flow to the vicinity of Grimes Mill where it would recross the river in an inverted siphon. This is the identical transfer system as described under the Caribou community system with increased hydraulic capacity. The gravity line would be increased to 48 inch. Residence time in the transfer system between Caribou and Grimes Mill will be about 1 hour.

Upon arrival at Grimes Mill, the combined wastewater would receive biological treatment prior to discharge to the river. Any industrial wastes generated at the Grimes Mill industrial park would also enter the plant.

The treatment plant at Grimes Mill for the combined wastes would be similar to that described for the Caribou community system, except for increased capacity. For high load conditions a four unit plant is suggested as shown on the general site plan, Figure 64. The medium load plant would be a three unit system, while the low load plant would have two units. As suggested for the Caribou community system, the land easterly of the site should be acquired for possible future installation of effluent holding ponds. The need for such units at Grimes Mill is discussed in the companion Regional Planning Report. The sizing of key treatment plant units for the three load ranges is presented in Table 86.

The above described regional Core Area system would provide treatment to meet current water quality standards on the Prestile Stream and Aroostook River. The impact of the treated effluent on the water quality of the Aroostook River is discussed in the companion River Basin Planning Report.

The second alternative for full regional interconnection in the Core Area consists of a treatment-transport system between Potato Service, Inc and Caribou. A full discussion of the studies undertaken to evaluate the treatment-transport were included in Section V under a separate subsection entitled "Treatment-Transport Studies".

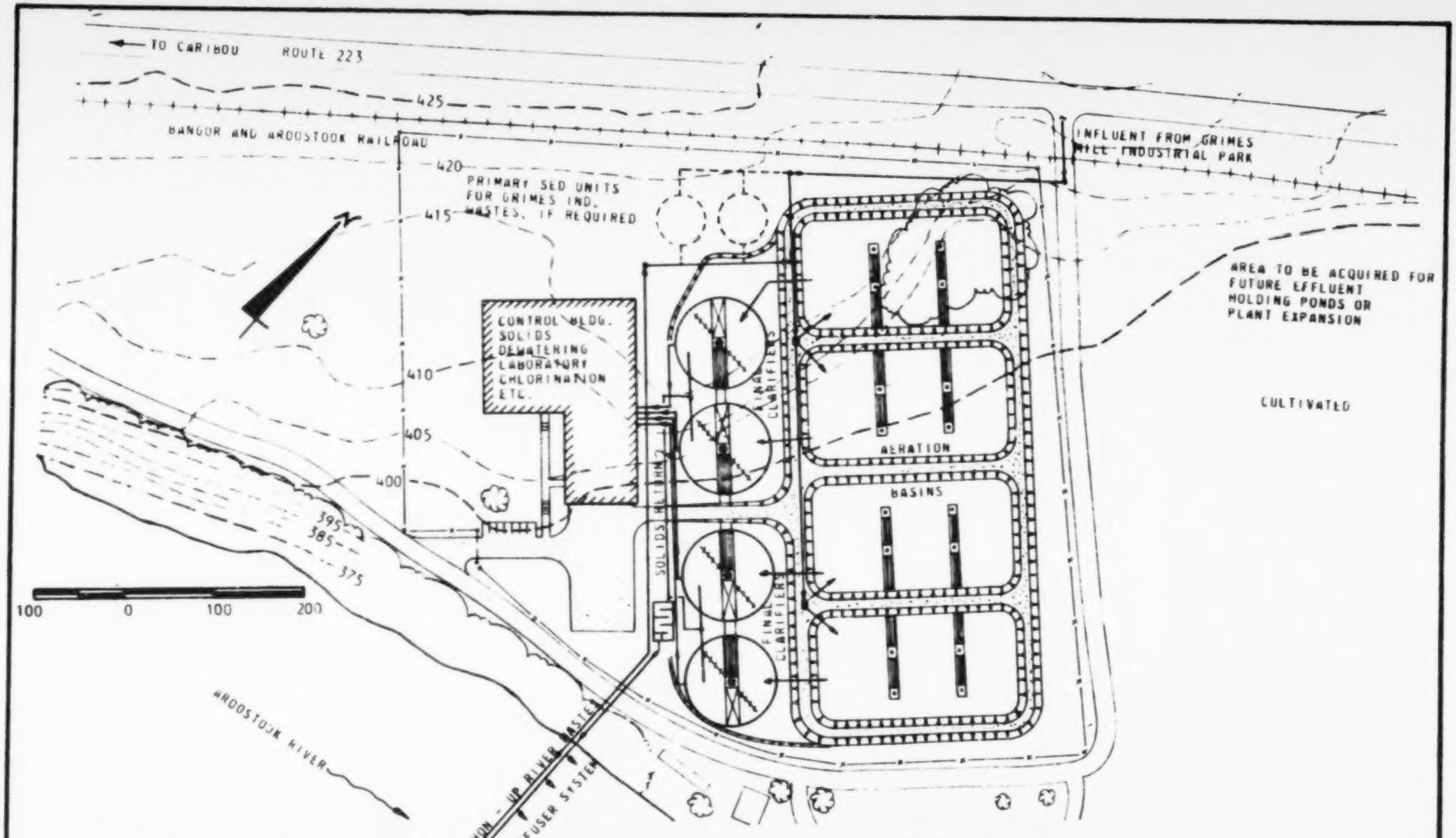

TO CARIBOU
ROUTE 223
425
BANGOR AND AROOSTOOK RAILROAD
INFLUENT FROM GRIMES MILL INDUSTRIAL PARK
420
PRIMARY SED UNITS FOR GRIMES IND. WASTES, IF REQUIRED
415
N
AREA TO BE ACQUIRED FOR FUTURE EFFLUENT HOLDING PONDS OR PLANT EXPANSION
CONTROL BLDG. SOLIDS DEWATERING LABORATORY CHLORINATION ETC.
410
FINAL CLARIFIERS
AERATION
BASINS
CULTIVATED
405
400
SOLIDS RETURN
395
385
375
100
0
100
200
AROOSTOOK RIVER
INV SIPHON - UP RIVER WASTES
OUTFALL DIFFUSER SYSTEM

FIGURE 64
PRELIMINARY SITE PLAN
GRIMES MILL TERMINAL PLANT - CORE AREA

TABLE 86

KEY TREATMENT PLANT SIZING - CARIBOU REGIONAL PLANT*

Loading Condition	Aeration Tank Vol CF	Secondary Sed Tank Surf Area	O_2 Req Lbs/hr	Excess Solids Lbs/day	Vac Filter A-Sq Ft	CL_2 Contact Vol CF	CL_2 Req Lbs/day
High	1,251,336	31,200	7,248	98,500	2,368	20,052	1,440
Medium	814,620	20,310	5,076	69,400	1,668	13,053	936
Low	425,454	10,608	1,852	24,600	590	6,818	489

*Sizing given is for full plant. For high load divide by 4 for individual unit sizes, for medium loading divide by 3, and for low loading divide by 2.

As previously discussed, the treatment-transport channel system, combined with terminal facilities at either Caribou or Grimes Mill, would provide a treatment system capable of handling the design loads from the Core Area in a manner that would meet current water quality standards on both the Prestile Stream and Aroostook River.

SOLIDS DISPOSAL

Treatment works proposed for either individual community systems or interconnected regional systems will generate significant quantities of excess solids which must be disposed of. This disposal must be accomplished in a manner which will not create additional environmental problems. There will be two basic components of the solids generated; namely, the solids from primary clarification units, and the excess biological solids generated in the activated sludge systems. Each has its own characteristics and disposal problems.

PRIMARY SOLIDS - Primary sedimentation facilities have been proposed for all potato processing wastes. The solids removed from such units are entirely a potato solid with definite value as an animal feed. This material, when dewatered, can be utilized directly as a cattle feed when mixed with other feed components, or it can be further dried and processed for shipping as a feed product. There is currently one plant in the area which dries and processes waste potato solids for feed. One processor uses a portion of this material for direct cattle feeding.

As additional primary clarification units are installed, the volume of this material will increase. Facilities for processing and utilizing this material as an animal feed product should keep pace with production. The only alternative to animal feed at this time is a sanitary landfill system. While the landfill may be suitable, utilization of the material as a by-product would seem somewhat more desirable.

EXCESS BIOLOGICAL SOLIDS - The biological treatment systems proposed for the area will generate significant quantities of excess biological solids which must be disposed of. This material will be the waste products and excess organisms from the biological mass. With maximum load conditions at all plants within the study area, from 65 to 75 tons, dry weight, could be generated within the systems. This material will be dewatered on vacuum filters to solid form and would represent 500 to 550 cubic yards per day during the processing season. These amounts would be lessened in general relation to the BOD for other loadings studied.

This material can present a significant disposal problem. The available disposal options are as follows:

1. Incineration
2. Sanitary landfill
3. Utilization on the land as a soil conditioner

Incineration of the excess solids in a multiple hearth system could be a reasonable disposal method if all solids were concentrated at one location, but would not be practical at each local unit. The system would be designed for solids loadings (dry weight basis) of 65 to 75 tons per day. With the dewatered biologic solids it is likely that auxilliary fuel would be required. It must also be noted that a residual ash must be handled from the incinerator and landfilled. The ash content of the biologic solids would be limited.

Studies conducted under the companion River Basin Planning Report suggest that utilization of the nutrient and organic values of the excess biological solids may be possible. It is felt, therefore, that installation of incineration facilities for excess biological solids disposal at this time is not warranted. However, if solids utilization does not materialize, installation at a future date could be considered.

Sanitary landfill would be considered a viable disposal method for the excess biological solids. The dewatered material will be volitile in nature and subject to additional bacterial breakdown. If a landfill operation is adopted, it may be desirable to utilize lime in the dewatering process to raise the pH of the dewatered solids to retard bacterial action.

Current solid waste disposal practices in the area consist primarily of open dumps. It is likely that air quality standards now under consideration by the State Environmental Improvement Commission will prohibit such methods in the near future. It is likely that by the time the biological treatment plants are constructed, the communities in the region are going to have operating municipal refuse sanitary landfills. It would appear logical to utilize joint landfill operations for municipal refuse and excess solids disposed from the treatment systems. The actual locations of such facilities must be selected based on land availability, soil conditions, and other factors. During this selection process the points of generation of the excess biological solids should be considered. If good landfill practices are adopted by the municipalities, disposal of the excess biological solids, in combination with the refuse, should not create any severe environmental problems.

The excess biologic solids will have a nutrient or fertilizer value. This material could be utilized as an ingredient in a manufactured fertilizer product. However, the quantity generated, and the difficulties in drying the material probably renders this usage economically unattractive. Studies under the Basin Planning Report, however, suggest the use of this material in its semi-dry form as a soil conditioner. The reader is referred to the Basin Planning Report for more detailed discussions. If such a use can be developed, a portion, or all, of the material being landfilled could be diverted to soil conditioning projects. Thus, this use factor would make adoption of sanitary landfill procedures in conjunction with municipal refuse disposal somewhat more attractive than incineration.

SECTION VIII

CAPITAL AND ANNUAL COSTS

The previous section presented preliminary designs for a wide range of pollution control alternatives for the Aroostook River-Prestile Stream Basins. Of the many alternatives presented, several provide systems which can meet current water quality standards. Full evaluation of the technically viable systems require an economic analysis of each, including consideration of capital costs, governmental aid programs, and operating costs. Such an analysis has been made on the basis of costs developed from preliminary design plans and data. It must be recognized that costs developed at this time are not based on firm working drawings and must be considered as preliminary costs only. However, they should be sufficient to realistically reflect the magnitude of the project cost requirements, and should provide data on which system comparisons can be made.

The construction cost estimates were based on unit prices applied to quantity estimates based on preliminary designs. The unit costs used were typical of those received in Maine for similar work in recent months. For the treatment-transport channel, consultation with contractors was used to aid in estimates. Quotations from equipment suppliers were obtained where applicable. An allowance of 25% for engineering, technical services and contingencies has been applied to all estimated construction costs. Land costs are not included in the estimates due to their wide variability with local conditions. In the overall project, however, land costs should not be an appreciable factor. In general, construction costs reflect an Engineering News Record index of about 1620. Future increases can be obtained by applying the index at the time of consideration.

The existence of State and Federal grant-in-aid programs will have substantial effect on local capital costs. The question of State and Federal grants-in-aid as applied to industrial waste plants and combined domestic-industrial waste treatment facilities has been in a state of flux over the past year. It is our understanding that current interpretation of Federal guidelines is as follows:

1. Plants treating only an industrial wastewater are not eligible for grants-in-aid, even if built and operated by a public authority, unless special circumstances can be demonstrated.
2. Those portions of a joint plant passing only industrial wastewater are not eligible for aid. However, those portions treating both wastes jointly are eligible if constructed and operated by a public agency, and the system provides general treatment to the entire waste load of the community. The public agency must also demonstrate by contract that the industry is paying its full share of both local capital and operating costs.

The cost analyses of this section are based on the following aid factors

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for those portions of the project deemed eligible under the general criteria outlined above.

1. State Aid = 30% of eligible project costs
2. Federal Aid, EPA = 50% of eligible project costs
3. Federal Aid = 5% additional for eligible project costs meeting regional planning requirements.

It should be noted that land or right of way acquisition and certain legal and administrative costs are not included in the eligible project cost.

The central Aroostook region is currently an area designated by the Economic Development Administration as eligible for grant funds for projects benefitting the economic development of the area. As a solution to the pollution control problems of the area is a key factor in its economic development program, some portions of the work may be eligible for additional aid. This could raise the total aid factor to 90%. However, for preliminary analysis we have used the 85% factor as a reasonable level for system comparisons.

INDIVIDUAL COMMUNITY SYSTEMS

WASHBURN AREA - CAPITAL COSTS - Capital cost estimates have been prepared for the interceptor and treatment plant alternatives presented in the foregoing sections for the Washburn area. The following estimates are keyed to the Washburn coding system presented in Table 66. The reader is referred thereto for definition of alternatives and loading factors. Estimated capital costs for treatment facilities in the Washburn area are presented in Table 87.

Analysis of Table 87 indicates that the costs of a plant treating the Taterstate waste only will range from about $1,445,000 to $1,980,000 depending on the configuration, processing plant expansion, and on in-plant changes undertaken. A plant treating both the domestic and the industrial wastewaters will have a cost range of about $1,600,000 to $2,314,000, again depending on configuration, plant expansion, and in-plant changes at Taterstate. These costs all include a river outfall and diffuser system, and the pumping facilities leading to the plant. The total cost of separate plants would be the sum of the Taterstate plant plus the municipal extended aeration plant. This total cost would range from $1,775,000 to about $2,450,000. A summary of the potential cost savings, assuming dual units at the Taterstate plant, are presented in Table 88. Dual units ar assumed for comparative purposes to obtain equal operating flexibility and plant redundancy.

From Table 88 it can be seen that a significant capital cost savings, ranging from $338,000 to $388,000 may be realized by installation of a joint treatment facility. The use of single units at a strictly industrial plant would lessen this savings slightly, but would not provide identical treatment flexibility.

TABLE 87

SUMMARY - CAPITAL COST ESTIMATES

Washburn

Condition C-1 - Single Unit
Taterstate Alone

Sub Condition	Est Cost
A-1 + B-1	$1,736,000
A-2 + B-1	1,629,000
A-3 + B-1	1,315,000
A-1 + B-2	1,960,000
A-2 + B-2	1,820,000
A-3 + B-2	1,445,000

Condition C-2 - Dual Units
Taterstate & Municipal Combined

Sub Condition	Est Cost
A-1 + B-1	$2,086,000
A-2 + B-1	1,919,000
A-3 + B-1	1,600,000
A-1 + B-2	2,314,000
A-2 + B-2	2,174,000
A-3 + B-2	1,737,000

Condition C-1 - Dual Units
Taterstate Alone

Sub Condition	Est Cost
A-1 + B-1	$1,980,000
A-2 + B-1	1,832,000
A-3 + B-1	1,518,000
A-1 + B-2	2,230,000
A-2 + B-2	2,060,000
A-3 + B-2	1,605,000

Condition C-3 - Dual Units
Municipal Alone

Sub Condition	Est Cost
20 hr Aer	$ 470,000
20 hr Aer	460,000

Note: See Table 66 for design condition coding summary

TABLE 88

COMPARATIVE COST SUMMARY - JOINT VS SEPARATE PLANTS

WASHBURN

Industry Loading	Capital Cost Separate Plants Condition C-1 & C-3	Joint Plant Condition C-1	Capital Cost Estimated Savings
A-1 + B-1	$2,450,000	$2,086,000	$364,000
A-2 + B-1	2,302,000	1,919,000	383,000
A-3 + B-1	1,988,000	1,600,000	388,000
A-1 + B-2	2,700,000	2,314,000	386,000
A-2 + B-2	2,530,000	2,174,000	356,000
A-3 + B-2	2,075,000	1,737,000	338,000

Note: See Table 66 for design condition coding summary

Federal and State grants-in-aid should be available to assist in the capital costs of the proposed Washburn facilities. The applicable aid factors and eligibility requirements would be as previously discussed. At Washburn, this interpretation will mean that if a separate plant is constructed for only the Taterstate wastewaters, it, in all likelihood, will not be eligible for aid. It definitely would not be eligible if constructed by Taterstate. It would also be assumed that a new primary sedimentation unit would not be eligible, but the remaining portion of the joint plant and joint outfall to the Aroostook River would probably be eligible for aid, if all other requirements are met. The effect of this aid interpretation has a very marked effect on local project costs. This is illustrated in Table 89.

TABLE 89

EFFECT OF GRANT-IN-AID PROGRAMS ON LOCAL COST

WASHBURN

SEPARATE PLANTS (Condition C-1 & C-3)

Industry Loading	Total Cost	Elig Cost	Est Aid	Estimated Local Cost
A-1 + B-1	$2,450,000	$ 470,000	$ 400,000	$2,050,000
A-2 + B-1	2,302,000	470,000	400,000	1,902,000
A-3 + B-1	1,988,000	470,000	400,000	1,588,000
A-1 + B-2	2,700,000	470,000	400,000	2,300,000
A-2 + B-2	2,530,000	470,000	400,000	2,130,000
A-3 + B-2	2,075,000	470,000	400,000	1,675,000

JOINT PLANT (Condition C-2)

Industry Loading	Total Cost	Elig Cost	Est Aid	Estimated Local Cost
A-1 + B-1	$2,086,000	$1,895,000	$1,610,000	$ 476,000
A-2 + B-1	1,919,000	1,735,000	1,472,000	447,000
A-3 + B-1	1,600,000	1,435,000	1,220,000	380,000
A-1 + B-2	2,314,000	2,109,000	1,790,000	524,000
A-2 + B-2	2,174,000	1,979,000	1,680,000	494,000
A-3 + B-2	1,737,000	1,559,000	1,323,000	414,000

NOTE: See Table 66 for design condition coding summary

As can be seen, the lack of aid for the separate Taterstate plant greatly increases the local capital cost, and would cause the local community to lean strongly to the joint plant if it, in turn, is determined eligible. However, it must be noted that, on a capital cost basis, a joint plant would be indicated regardless of aid eligibility.

WASHBURN AREA - OPERATING COSTS - Any comparative analysis must consider operational and maintenance costs as well as capital costs, especially as all operating costs must be raised locally. Preliminary estimates of operating costs have been prepared for both a joint plant and separate plant systems. The estimates of operational costs are summarized in Table 90.

TABLE 90

SUMMARY OF OPERATING COSTS

WASHBURN

	Separate Plants				
Industry Loading	Taterstate Condition C-1	Town Condition C-3	Total	Joint Plant Condition C-3	Estimated Savings
A-1 + B-1	$70,000/yr	$18,000/yr	$88,100/yr	$75,100/yr	$13,100/yr
A-2 + B-1	65,200	18,000	83,200	69,700	13,500
A-3 + B-1	53,000	18,000	71,000	57,000	14,000
A-1 + B-2	76,500	18,000	94,500	82,800	11,700
A-2 + B-2	72,100	18,000	90,100	76,800	13,300
A-3 + B-2	57,000	18,000	75,000	60,300	14,700

NOTE: See Table 66 for design condition coding summary

The costs in Table 90 include labor costs, power costs, chemical costs, heating and general utilities and maintenance. The major items are labor and power costs. Table 90 indicates that, in addition to a capital cost savings, an operating cost savings of about $13,500/yr may be possible with a joint plant. These savings are mostly in the form of labor costs.

WASHBURN AREA - ANNUAL REVENUE REQUIREMENTS - A total cost comparison of the systems studied can be achieved by estimating the finance costs for the capital system and combining them with the operating costs to arrive at a total annual revenue requirement for the system. For this preliminary analysis, we have assumed 30 year term bonds at 6%. While such bonding procedures may or may not be acutally utilized by the Town or the industry, the costs thus arrived at represent reasonable average finance costs for the project. Prior to construction, the Town should seek expert financial advice on optimum bonding procedures for the project.

The summary of estimated finance costs is presented in Table 91, while the estimated total annual costs are summarized in Table 92.

TABLE 91

SUMMARY OF LOCAL FINANCE COSTS*

WASHBURN

Industry Loading	Separate Plants Condition C-1 & C-3	Joint Plant Condition C-2	Savings
A-1 + B-1	$149,000/yr	$34,500/yr	$114,500/yr
A-2 + B-1	137,400	32,200	105,200
A-3 + B-1	115,200	27,700	87,500
A-1 + B-2	167,300	37,900	129,400
A-2 + B-2	154,700	35,700	119,000
A-3 + B-2	126,200	29,900	96,300

*Based on Dual Unit System for all configurations.

NOTE: See Table 66 for design condition coding summary

TABLE 92

SUMMARY - TOTAL ANNUAL REVENUE REQUIREMENTS*

WASHBURN

Industry Loading	Separate Plants Condition C-1 & C-3	Joint Plant Condition C-2	Savings
A-1 + B-1	$237,100/yr	$109,600/yr	$127,500/yr
A-2 + B-1	220,500	101,900	118,600
A-3 + B-1	186,200	84,700	101,500
A-1 + B-2	271,800	120,700	151,100
A-2 + B-2	244,800	112,500	132,300
A-3 + B-2	201,200	90,200	111,000

*Based on Dual Unit Systems for all configurations.

NOTE: See Table 66 for design condition coding summary

Study of local treatment at Washburn indicates that the Town and Tater-state Frozen Foods should consider joint treatment of domestic and industrial wastes in the community. The potential economic advantages to both parties far outweigh the administrative and financial negotiation problems inherent in such a system. This recommendation is in line with prior recommendations of the Jordan Company in its 1966 Report[3] to the Town of Washburn. The following advantages are realized with a joint system:

1. Total capital costs of joint treatment plant are about 15% less than two plants, if equally flexible and reliable systems are installed
2. Governmental aid would not be available for a plant treating only the Taterstate waste. This has a very large impact on local costs.
3. Operational costs for a joint plant are also about 15% less than for the two plant system.
4. The additional nutrients provided to a joint system by the municipal wastes will be beneficial in operating the biological system.

Future expansion of the treatment plant must also be considered. The foregoing preliminary design and cost estimates are based on a municipal waste load projection to the year 1995. The loads generated at Taterstate are assumed to represent present production (with in-plant changes considered), and with a planned expansion of a flake line. Should Taterstate greatly increase its production beyond those projected, or should another significant waste producing industry locate in Washburn, the plant would have to be expanded. While it is practical to project municipal waste loads, it is not practical to anticipate significant industrial expansion unless it is firmly planned. However, in an industrial treatment facility, it is vital to design the initial facility so that it may be expanded with relative ease. This approach has been taken at Washburn. If it becomes the public policy of the Town or other public agency to provide treatment of the Taterstate and other industry wastes, it will be no more difficult, and probably less so, to expand a joint plant than it would a separate plant. Provision of such facilities will, in fact, enhance the Town's economic growth potential.

In addition to the treatment costs indicated in prior tables, the Town must support the remaining in-town interceptor system, if it is constructed at this time. The estimated total and local cost of these additional facilities is presented in Table 93.

TABLE 93

CAPITAL COST ESTIMATES - SUPPLEMENTAL TOWN FACILITIES

WASHBURN

Unit	Total Cost	Local Cost*
Interceptors	$ 99,000	$ 14,800
Storm Drains	70,000	70,000
Crouseville Int & TP	70,000	10,500
Crouseville San Sewers	65,000	32,500
Sanitary Sewer Extension	70,000	35,000
TOTALS	$374,000	$162,800

*Assumes 85% aid on Int & TP, State & EPA, - 50% FHA aid on Sanitary Sewers.

MAPLETON AREA COSTS - The scope of this study generally would not include cost analysis of each small community in the Basin. However, as the geographical orientation of Mapleton is such that physical interconnection with the Core Area could be feasible, cost data was prepared to allow realistic comparison of local versus regional system.

Capital cost estimates for the Mapleton treatment plant are available through the Town's 1967 planning effort. The capital cost of the treatment plant at that time was estimated at $190,240, including engineering and contingencies. This estimate was based on an Engineering News Record Index of 1100. Updating to 1971 prices, with an ENR Index of 1600, this estimate would rise to $277,000. The 1967 design did not provide facilities for excess solids handling during the winter months. Current regulations will make such facilities mandatory. Inclusion of solids handling facilities will raise the estimated plant cost to about $320,000. The local cost of this plant would be about $48,000, again assuming an 35% aid factor. Operating costs at Mapleton are estimated at about $12,000 per year. The comparison of local costs at Mapleton, with regional interconnection, is presented in a later section.

PRESQUE ISLE AREA - CAPITAL COSTS - Cost estimates have been prepared for the local community system in the Presque Isle area. The facilities estimated include treatment facilities for the Presque Isle domestic wastes and the Potato Service, Inc industrial wastes both individually and in combination. For the combined system the costs of transporting the domestic wastewater to the joint plant site have also been estimated.

The loading condition designation is that described earlier in Table 70. The reader is referred thereto for definition. The capital cost estimates are summarized in Table 94.

TABLE 94

SUMMARY CAPITAL COSTS

Presque Isle Area

Individual Systems

Industry Loading	PSI Alone Condition B-1 Est Cost	PQI Alone Condition B-3 Est Cost	Total Est Cost
A-1	$4,400,000	$1,100,000	$5,500,000
A-2	3,950,000	1,100,000	5,050,000
A-3	2,650,000	1,100,000	3,750,000

TABLE 94

(CONTINUED)

Joint System (Condition B-2)

Industry Loading	Transport Cost	Adj In-town Plant Cost	Treatment Cost	Total Cost
A-1	$260,000	$190,000	$4,470,000	$4,920,000
A-2	260,000	190,000	4,100,000	4,550,000
A-3	260,000	190,000	2,850,000	3,300,000

NOTE: See Table 70 for design condition coding summary

Analysis of Table 94 indicates considerable capital cost savings can be gained by adopting a joint treatment program for the Presque Isle domestic wastes and the Potato Service, Inc industrial wastewaters. The potential savings are summarized in Table 95.

TABLE 95

SUMMARY - JOINT SYSTEM CAPITAL COST SAVINGS

CONDITION B-2

Presque Isle Area

Loading Industry	Capital Cost Savings
A-1	$500,000
A-2	500,000
A-3	450,000

NOTE: See Table 70 for design condition coding summary

The local share of the capital costs reported above will be lessened by application of State and Federal aid programs, as described in the introduction to this section. The effect of this aid is illustrated in Table 96.

Analysis of Table 96 indicates that application of Federal and State aid factors to the capital costs greatly increases the local cost advantage of the joint system versus individual systems. These local cost savings are summarized in Table 97.

TABLE 96

LOCAL CAPITAL COSTS

Presque Isle Area

Individual Systems

Industry Loading	PSI Alone Condition B-1 Est Local Cost	PQI Alone Condition B-3 Est Local Cost	Total Est Local Cost
A-1	$4,400,000	$165,000	$4,565,000
A-2	3,950,000	165,000	4,115,000
A-3	2,650,000	165,000	2,815,000

Joint System - Condition B-2

Industry Loading	Transport Est loc Cost	Adj In-town Plant Est Local Cost	Joint Treatment Est Local Cost	Total Est Local Cost
A-1	$39,000	$28,500	$670,500	$738,000
A-2	39,000	28,500	615,500	683,000
A-3	39,000	28,500	428,500	496,000

NOTE: See Table 70 for design condition coding summary

TABLE 97

SUMMARY - LOCAL CAPITAL COST SAVINGS - JOINT SYSTEM

CONDITION B-2

Presque Isle Area

Loading Industry	Capital Cost Savings
A-1	$3,827,000
A-2	3,432,000
A-3	2,319,000

NOTE: See Table 70 for design condition coding summary

PRESQUE ISLE AREA - OPERATING COSTS - Estimates have been prepared for the operating costs of both the individual and combined plants in the Presque Isle area. These costs include labor, power, chlorine and other chemicals, and general operation and maintenance. These costs assume a 10 month operation at Potato Service, Inc. Table 98 summarizes the operating costs of the required facilities.

TABLE 98

SUMMARY OPERATING COSTS

Presque Isle Area

Individual System

Industry Loading	PSI Alone Condition B-1 Oper Costs	PQI Alone Condition B-3 Oper Costs	Total Oper Cost
A-1	$185,000/yr	$68,000/yr.	$253,000/yr
A-2	168,000	68,000	236,000
A-3	118,000	68,000	186,000

Joint System - Condition B-2

Industry Loading	Total Oper Cost
A-1	$243,000/yr
A-2	230,000
A-3	176,000

NOTE: See Table 70 for design condition coding summary

Analysis of Table 98 indicates a modest operating cost savings by adoption of a joint plant. The savings are not as pronounced as was the case in Washburn due to the transportation costs and the costs of general maintenance at the existing Presque Isle facility.

PRESQUE ISLE AREA - ANNUAL REVENUE REQUIREMENTS - The total annual costs required to support the Presque Isle area facilities will be the operation costs, as outlined above, plus the finance costs necessary to support the local bonds. Again, 30 year term bond at 6% have been assumed for preliminary design analysis. It is recognized, however, that industry would utilize other financial procedures if individual plants were adopted. Table 99 summarizes the finance costs, and Table 100 summarizes the total annual revenue required.

TABLE 99

SUMMARY - SYSTEM FINANCE COSTS

Presque Isle Area

Individual Systems

Industry Loading	PSI Alone Condition B-1 Finance Cost	PQI Alone Condition B-3 Finance Cost	Total Finance Cost
A-1	$320,000/yr	$12,000/yr	$332,000/yr
A-2	287,000	12,000	209,000
A-3	193,000	12,000	205,000

Joint System - Condition B-2

Industry Loading	Total Finance Cost
A-1	$54,500/yr
A-2	50,200
A-3	35,600

NOTE: See Table 70 for design condition coding summary

TABLE 100

SUMMARY - ANNUAL REVENUE REQUIREMENTS

Presque Isle Area

Individual Systems

Industry Loading	PSI Alone Condition B-1 Annual Cost	PQI Alone Condition B-3 Annual Cost	Total Annual Cost
A-1	$505,000/yr	$80,000/yr	$585,000/yr
A-2	455,000	80,000	535,000
A-3	311,000	80,000	391,000

Joint System - Condition B-2

Industry Loading	Total Annual Cost
A-1	$297,500/yr
A-2	280,200
A-3	231,600

NOTE: See Table 70 for design condition coding summary

Analysis of Table 100 indicates a substantial reduction in annual revenue required by adoption of the joint treatment system. Most of this savings is in the finance cost, which reflects both the overall capital cost reductions by a joint plant and the aid eligibility for public systems. The overall annual savings is summarized in Table 101.

TABLE 101

SUMMARY - ANNUAL COST SAVINGS - JOINT SYSTEM

CONDITION B-2

Presque Isle

Industry Loading	Annual Cost Savings
A-1	$287,500/yr
A-2	254,800
A-3	159,400

NOTE: See Table 70 for design condition coding summary

Study of local treatment at Presque Isle indicates that the Sewer District, or other public body, and Potato Service, Inc should consider joint treatment of domestic and industrial wastes in the community. The potential economic advantages to both parties far outweigh the administrative and financial negotiation problems inherent in such a system. The following are advantages of a joint system:

1. Total capital costs of joint treatment plant are about 10 to 12% less than two plants, if equally flexible and reliable systems are to be installed.
2. Governmental aid would not be available for a plant treating only the Potato Service, Inc waste. This has a very large impact on local costs.
3. Operational costs for a joint plant are also about 5 to 8% less than for the two plant system.
4. The additional nutrients provided to a joint system by the municipal wastes will be beneficial in operating the biological system.

Future expansion of the treatment plant must also be considered. The foregoing preliminary design and cost estimates are based on a municipal waste load projection to the year 1995. The loads generated at Potato Service, Inc are assumed to represent present production (with in-plant changes considered). Should Potato Service, Inc greatly increase its production beyond those projected, or should another significant waste producing industry locate in Presque Isle, the plant would have to be expanded. While it is practical to project municipal waste loads, it is not practical to anticipate significant industrial expansion unless it is firmly planned. However, in an industrial treatment facility, it is vital to design the initial facility so that it may be expanded with relative ease. This approach has been taken at Presque Isle-Potato Service, Inc joint plant. If it becomes public policy to provide treatment of the Potato Service, Inc and other industry wastes, it will be no more difficult, and probably less so, to expand a joint plant than it would a separate plant.

While the plant expansion in the vicinity of Potato Service, Inc would not be difficult, it must be recognized that the piped system from the existing Presque Isle plant to the vicinity of Potato Service, Inc will be an expansion constraint. If a major wet process industry should locate in in-town Presque Isle, the piped transport system could prove inadequate. However, the system could handle reasonably large loads by utilizing the existing sedimentation tanks as equalization tanks to remove peak flow excess. In general, however, it would be more desirable to locate such wet process industries in an area where direct flows to the municipal-industrial plant could be achieved. This factor should not be considered an obstacle to joint facilities, however, as it must be recognized that with in-town plant expansion, very definite capacity limits would exist.

CARIBOU AREA - CAPITAL COSTS - Cost estimates have been prepared for the local community systems in the Caribou area. In Caribou, current policy dictates joint treatment of the industrial and domestic wastewaters. All facilities, therefore, have been based on this policy, as described under the preliminary design section. Estimates have been prepared for the various options of in-town primary treatment, and the option of in-town expansion versus treatment of Grimes Mill. The loading condition and combination options are coded in the same manner as designated under Table 73. The reader is referred to that section for coding definitions. The capital costs for the expanded Caribou in-town (Condition D-1) facilities are presented in Table 102, and those of the Grimes Mill option (Condition D-2) are presented in Table 103.

Analysis of Tables 102 and 103 indicate that it will cost more to treat the Caribou wastewaters at Grimes Mill than by expansion of the in-town plant. The cost differential is summarized in Table 104.

TABLE 102

CAPITAL COSTS - IN-TOWN PLANT EXPANSION

CONDITION D-1

Caribou Area

WITHOUT GRIMES MILL FLOW (Condition C-1):

Loading Condition	Treatment Primary	Treatment Biological	Inter-ceptor	Est Total Capital Costs
A-1 + B-1 + C-1	-	$4,820,000	$175,000	$4,995,000
A-2 + B-1 + C-1	-	3,730,000	175,000	3,905,000
A-3 + B-1 + C-1	-	2,820,000	175,000	2,995,000
A-1 + B-2 + C-1	$115,000	4,660,000	-	4,775,000
A-2 + B-2 + C-1	90,000	3,620,000	-	3,710,000
A-3 + B-2 + C-1	75,000	2,810,000	-	2,885,000

WITH GRIMES MILL FLOW (Condition C-2):

Loading Condition	Treatment Primary	Treatment Biological	Inter-ceptor	Est Total Capital Costs
A-1 + B-1 + C-2	-	$5,940,000	$905,000	$6,845,000
A-2 + B-1 + C-2	-	4,760,000	905,000	5,660,000
A-3 + B-1 + C-2	-	3,920,000	905,000	4,825,000
A-1 + B-2 + C-2	$115,000	5,690,000	730,000	6,535,000
A-2 + B-2 + C-2	90,000	4,570,000	730,000	5,390,000
A-3 + B-2 + C-2	75,000	3,910,000	730,000	4,715,000

NOTE: See Table 73 for design condition coding summary

TABLE 103

CAPITAL COSTS - GRIMES MILL SITE

CONDITION D-2

Caribou Area

W/O MAJOR INDUSTRY AT GRIMES (Condition C-1)

Loading Condition	Treatment In-town	Treatment Biological	Transport Cost	Est Total Capital Costs
A-1 + B-1 + C-1	$675,000	$4,610,000	$930,000	$6,215,000
A-2 + B-1 + C-1	660,000	3,600,000	900,000	5,160,000
A-3 + B-1 + C-1	625,000	2,920,000	850,000	4,395,000
A-1 + B-2 + C-1	615,000	4,425,000	930,000	5,970,000
A-2 + B-2 + C-1	600,000	3,465,000	900,000	4,965,000
A-3 + B-2 + C-1	575,000	2,640,000	850,000	4,065,000

WITH INDUSTRY AT GRIMES (Condition C-2)

Loading Condition	Treatment In-town	Treatment Biological	Transport Cost	Est Total Capital Cost
A-1 + B-1 + C-2	$675,000	$5,650,000	$930,000	$7,255,000
A-2 + B-1 + C-2	660,000	4,540,000	900,000	6,100,000
A-3 + B-1 + C-2	625,000	3,700,000	850,000	5,175,000
A-1 + B-2 + C-2	615,000	5,390,000	930,000	6,935,000
A-2 + B-2 + C-2	600,000	4,340,000	900,000	5,840,000
A-3 + B-2 + C-2	575,000	3,685,000	850,000	5,110,000

NOTE: See Table 73 for design condition coding summary

TABLE 104

CAPITAL COST DIFFERENTIAL - IN-TOWN VS GRIMES SITE

Caribou Area

Loading Condition	Added Cost Grimes Site
A-1 + B-1 + C-1	$1,220,000
A-2 + B-1 + C-1	1,255,000
A-3 + B-1 + C-1	1,400,000
A-1 + B-2 + C-1	1,195,000
A-2 + B-2 + C-1	1,255,000
A-3 + B-2 + C-1	1,180,000
A-1 + B-1 + C-2	410,000
A-2 + B-1 + C-2	440,000
A-3 + B-1 + C-2	350,000
A-1 + B-2 + C-2	400,000
A-2 + B-2 + C-2	450,000
A-3 + B-2 + C-2	395,000

NOTE: See Table 73 for design condition coding summary

Table 104 indicates that if no significant industrial wastes are to be generated in the Grimes Mill industrial park (C-1 Conditions), it will cost about 1.25 million dollars additional to utilize the Grimes site. If, however, a significant wet process industry were to locate in the industrial park, this differential would drop to about $400,000. Other, non cost factors will also influence the site selection at Caribou. These are discussed in a later section.

The above tables also suggest that it will be more economic to install additional primary sedimentation facilities to handle the total waste flow at the in-town Caribou plant (Condition B-2), rather than separating the American Kitchen Foods process wastes by construction of a new relief interceptor (Condition B-1). The differential is about $200,000.

The local share of the Caribou system will be lessened by application of State and Federal aid programs as described in the introduction of this section. The effect of this aid is illustrated on Tables 105 and 106.

TABLE 105

LOCAL CAPITAL COSTS - IN-TOWN PLANT

CONDITION D-1

Caribou Area

WITHOUT GRIMES MILL FLOW (Condition C-1)

Loading Condition	Total Capital Cost	Local Cost
A-1 + B-1 + C-1	$4,995,000	$750,000
A-2 + B-1 + C-1	3,905,000	575,000
A-3 + B-1 + C-1	2,995,000	450,000
A-1 + B-2 + C-1	4,775,000	700,000
A-2 + B-2 + C-1	3,710,000	540,000
A-3 + B-2 + C-1	2,885,000	435,000

WITH GRIMES MILL FLOW (Condition C-2)

Loading Condition	Total Capital Cost	Local Cost
A-1 + B-1 + C-2	$6,845,000	$1,030,000
A-2 + B-1 + C-2	5,660,000	850,000
A-3 + B-1 + C-2	4,825,000	725,000
A-1 + B-2 + C-2	6,535,000	985,000
A-2 + B-2 + C-2	5,390,000	810,000
A-3 + B-2 + C-2	4,715,000	710,000

TABLE 106

LOCAL CAPITAL COSTS - GRIMES SITE

CONDITION D-2

Caribou Area

WITHOUT GRIMES MILL FLOW (Condition C-1)

Loading Condition	Total Capital Cost	Local Cost
A-1 + B-1 + C-1	$6,215,000	$930,000
A-2 + B-1 + C-1	5,160,000	770,000
A-3 + B-1 + C-1	4,395,000	670,000
A-1 + B-2 + C-1	5,970,000	800,000
A-2 + B-2 + C-1	4,965,000	745,000
A-3 + B-2 + C-1	4,065,000	585,000

WITH GRIMES MILL FLOW (Condition C-2)

Loading Condition	Total Capital Cost	Local Cost
A-1 + B-1 + C-2	$7,255,000	$1,075,000
A-2 + B-1 + C-2	6,100,000	920,000
A-3 + B-1 + C-2	5,175,000	775,000
A-1 + B-2 + C-2	6,935,000	1,045,000
A-2 + B-2 + C-2	5,840,000	865,000
A-3 + B-2 + C-2	5,110,000	760,000

NOTE: See Table 73 for design condition coding summary

Tables 105 and 106 indicate that the cost differential between the Grimes Mill site and the in-town expansion is greatly reduced by application of aid factors. If a major industry were to locate in the Grimes Mill industrial park, the added local cost would be about $30,000 to $40,000.

A third site alternative in the Caribou area which would be applicable if a major industry located in Grimes Mill would be the construction of two plants, one in-town to service existing loads, and a second plant at Grimes Mill to serve the new industry. A cost comparison has been prepared for this option and is presented in Table 107.

TABLE 107

CAPITAL COST COMPARISON - TWO PLANT OPTION

Caribou Area

Loading Condition	In-town Plant	Grimes Plant	Total	Joint Plant at Grimes	Savings Joint Plant
A-1 + B-1 + C-1	$4,995,000	$2,500,000	$7,495,000	$7,255,000	$240,000
A-2 + B-1 + C-1	3,905,000	2,500,000	6,405,000	6,110,000	295,000
A-3 + B-1 + C-1	2,995,000	2,500,000	5,495,000	5,175,000	320,000
A-1 + B-2 + C-1	4,775,000	2,500,000	7,275,000	6,935,000	340,000
A-2 + B-2 + C-1	3,710,000	2,500,000	6,210,000	5,848,000	370,000
A-3 + B-2 + C-1	2,885,000	2,500,000	5,385,000	5,150,000	275,000

NOTE: See Table 73 for design condition coding summary

Table 107 indicates that if a major industry were to locate at the Grimes Mill industrial park, it would be more economic to treat its wastewater jointly with the other Caribou area wastes than it would to build two plants. This advantage would be greatly increased if the new industry plant at Grimes Mill was not eligible for State and Federal aid. All further analysis at Caribou assumed the one plant system, either in-town, or at the Grimes site.

CARIBOU AREA - OPERATING COSTS - Estimates have been made of the operation costs for both plant sites in the Caribou area. All operating costs assume use of the expanded primary option (Condition B-2) rather than the new interception option (Condition B-1). It was further assumed that the Grimes site would be used only if a major industry were expected to locate there (Condition C-2). These costs include labor, power, chlorine and other chemicals, and general operation and maintenance costs. Again a 10 month industry operation season is assumed for the processing plants.

TABLE 108

SUMMARY - OPERATING COSTS

Caribou Area

In-town Plant Loading Condition D-1	Est Oper Cost
A-1 + B-2 + C-1	$258,000/yr
A-2 + B-2 + C-1	206,000
A-3 + B-2 + C-1	179,000
A-1 + B-2 + C-2	326,000
A-2 + B-2 + C-2	276,000
A-3 + B-2 + C-2	242,000

Grimes Plant Loading Condition D-2	Est Oper Cost
A-1 + B-2 + C-2	$363,000/yr
A-2 + B-2 + C-2	311,000
A-3 + B-2 + C-2	275,000

NOTE: See Table 73 for design condition coding summary

Analysis of Table 108 indicates that the operating cost at the Grimes Mill site would be about 10 to 15% more than the in-town site. This is due to the pumping costs required to carry the in-town wastewater to Grimes, and the fact that even if the biological plant is built at Grimes, the in-town plant must be maintained and operated as a primary plant with solids dewatering and disposal. Other, non-cost, factors concerning site selection in Caribou are discussed in a later section.

CARIBOU AREA - ANNUAL REVENUE REQUIREMENTS - The total annual costs required to support the Caribou area facilities will be the operating costs outlined above, plus the finance costs necessary to support the local bonds issued. Like the analysis of other community systems, a 30 year term bond at 6% interest has been assumed for preliminary design analysis. Table 109 summarizes the local annual finance costs and the total annual revenue requirements.

TABLE 109

SUMMARY - FINANCE COSTS & ANNUAL REVENUE REQUIREMENTS

Caribou Area

IN-TOWN PLANT (Condition D-1)

Loading Condition	Est Finance Cost	Est Total Annual Rev Req
A-1 + B-2 + C-1	$51,000/yr	$309,000/yr
A-2 + B-2 + C-1	40,000	246,000
A-3 + B-2 + C-1	32,000	211,000
A-1 + B-2 + C-2	72,000	398,000
A-2 + B-2 + C-2	59,000	335,000
A-3 + B-2 + C-2	52,000	294,000

GRIMES SITE (Condition D-2)

Loading Condition	Est Finance Cost	Est Total Annual Rev Req
A-1 + B-2 + C-2	$76,000/yr	$429,000/yr
A-2 + B-2 + C-2	64,000	375,000
A-3 + B-2 + C-2	56,000	331,000

Analysis of Table 109 indicates that use of the Grimes site would cost about $30,000 to $40,000 per year more than the in-town site, assuming a major industry locates in the Grimes Mill industrial park. However, other factors must be considered in final site selection in the Caribou area. The main problem faced with expansion of the in-town Caribou site is the limited space available. This is contrasted by the Grimes site which has ample land area available. This space factor will affect the site selection decision in several ways.

The in-town site can be utilized to treat the expected wastewaters flows, including flow from a major industry in the Grimes Mill industrial park, to a level sufficient to meet current water quality standards. However, as discussed in the companion Basin Planning Report, the assimilative capacity of the Aroostook River will be fully taxed at times of drought conditions. This raises the distinct possibility of having to curtail waste load input on occasions. This can most effectively be accomplished by diverting a portion of the treated waste flow to a storage pond during drought for release at a higher flow regime in the river. It is quite possible that such facilities may be eventually required if industrial expansion is to take place. There is no opportunity to provide such a facility at the in-town plant location, while there is sufficient space at the Grimes site.

A second factor which must be considered in site selection at Caribou is the possibility of the requirement of nutrient removal at some future date. The fact that the Aroostook River is a trans-boundary river tributary to the Saint John may hasten this requirement due to the power pools on the Saint John at Beechwood and Mactaquac. A Saint John River Basin study is expected to begin shortly which may clarify this issue. It should be noted that the limited space at the in-town site would make installation of such facilities somewhat more costly and difficult than at the Grimes site.

A third factor to be considered in site selection at Caribou is that of future solids handling requirements. The initial program will call for sanitary landfill of the excess biologic solids. This can be accomplished from either site. However, the companion Basin Planning Report recommends that a program be established to attempt to utilize all, or a portion of, the excess biologic solids as a soil conditioner on agricultural lands. To accomplish this, some storage capacity for excess solids on site would be desirable. This cannot be obtained at the in-town location because of site limitations and its proximity to highly developed areas. Such storage or processing of solids in the future could be accomplished at the Grimes Mill site.

In summary, while the initial costs of treatment would seem to favor plant expansion at the in-town site, long range space and treatment requirements may offset this short range cost advantage. If a local community system is adopted at Caribou, serious consideration should be given to use of the Grimes Mill site.

FORT FAIRFIELD AREA - CAPITAL COSTS - Capital cost estimates have been prepared for the treatment plant alternatives at Fort Fairfield as proposed in the foregoing sections. The following estimates are keyed to the Fort Fairfield coding system presented earlier in Table 75. The reader is referred thereto for definition of alternatives and loading factors. Estimated capital costs for the treatment facilities at Fort Fairfield are presented in Table 110.

TABLE 110

SUMMARY - CAPITAL COST ESTIMATES

Fort Fairfield

Condition C-1 - Single Unit A & P Alone Industry Loading	 Est Cost	Condition C-2 - Dual Units A & P and Municipal Combined Industry Loading	 Est Cost
A-1 + B-1	$1,400,000	A-1 + B-1	$1,520,000
A-2 + B-1	1,390,000	A-2 + B-1	1,480,000
A-3 + B-1	1,050,000	A-3 + B-1	1,210,000
A-1 + B-2	1,290,000	A-1 + B-2	1,480,000
A-2 + B-2	1,280,000	A-2 + B-2	1,440,000
A-3 + B-2	1,050,000	A-3 + B-2	1,170,000

TABLE 110
(CONTINUED)

Condition C-1 -Dual Units A & P Alone Industry Loading	 Est Cost
A-1 + B-1	$1,488,000
A-2 + B-1	1,460,000
A-3 + B-1	1,120,000
A-1 + B-2	1,420,000
A-2 + B-2	1,350,000
A-3 + B-2	1,110,000

Condition C-3 - Dual Units Municipal Alone Sub Condition	 Est Cost
24 hr Aer	$ 500,000
20 hr Aer	490,000

NOTE: See Table 75 for design condition coding summary

Analysis of Table 110 indicates that the costs of a plant treating the A & P waste only would range from about $1,050,000 to $1,480,000 depending on the configuration and on in-plant changes undertaken. A plant treating both the domestic and industrial wastewaters will have a cost range of from about $1,170,000 to $1,520,000, again depending on configuration and in-plant changes at A & P. The total cost of separate plants would be the sum of the A & P plant plus the municipal extended aeration plant. This total cost would range from about $1,530,000 to about $1,980,000. The potential capital cost savings, however, must be tempered by an added capital cost of about $60,000 to transfer the domestic wastewaters to the A & P site. A summary of the potential cost savings, assuming dual units at the A & P plant, is presented in Table 111.

TABLE 111

COMPARATIVE COST SUMMARY - JOINT VS SEPARATE PLANTS

Fort Fairfield

Industry Loading	Capital Cost Separate Plants Condition C-1 & C-3	Capital Cost Joint Plant Condition C-2	Estimated Savings
A-1 + B-1	$1,980,000	$1,580,000	$400,000
A-2 + B-1	1,960,000	1,540,000	420,000
A-3 + B-1	1,620,000	1,270,000	350,000
A-1 + B-2	1,920,000	1,540,000	380,000
A-2 + B-2	1,850,000	1,500,000	350,000
A-3 + B-2	1,510,000	1,230,000	380,000

NOTE: See Table 75 for design condition coding summary

From Table 111 it can be seen that a significant capital cost savings may be realized by installation of a joint treatment facility, ranging from $350,000 to $420,000. The existance of State and Federal grant-in-aid programs will have substantial effect on local capital costs. Eligibility for State and Federal grants-in-aid, as applied to industrial waste plants and combined domestic-industrial waste treatment facilities was discussed in earlier paragraphs.

At Fort Fairfield, this interpretation will mean that if a separate plant is constructed for the A & P wastewaters, it would not be eligible for aid. It definitely would not be eligible if constructed by the A & P Company. Also, a new primary sedimentation tank and influent pumps for the A & P processing wastes would not be eligible for aid, even if a joint plant is constructed. The remaining portion of the joint plant would be eligible for aid, if all other requirements are met. The effect of this aid interpretation has a very marked effect on local project costs. This is illustrated in Table 112.

TABLE 112

EFFECT OF GRANT-IN AID PROGRAMS ON LOCAL COST

Fort Fairfield

SEPARATE PLANTS (Condition C-1 and C-3)

Industry Loading	Total Cost	Elig Cost	Est Aid	Estimated Local Cost
A-1 + B-1	$1,980,000	$ 500,000	$ 425,000	$1,555,000
A-2 + B-1	1,960,000	500,000	425,000	1,535,000
A-3 + B-1	1,620,000	500,000	425,000	1,195,000
A-1 + B-2	1,920,000	500,000	425,000	1,495,000
A-2 + B-2	1,850,000	500,000	425,000	1,435,000
A-3 + B-2	1,610,000	500,000	425,000	1,185,000

JOINT PLANT (Condition C-2)

Industry Loading	Total Cost	Elig Cost	Est Aid	Estimated Local Cost
A-1 + B-1	$1,580,000	$1,480,000	$1,260,000	$ 320,000
A-2 + B-1	1,540,000	1,450,000	1,235,000	305,000
A-3 + B-1	1,270,000	1,200,000	1,020,000	250,000
A-1 + B-2	1,540,000	1,440,000	1,225,000	315,000
A-2 + B-2	1,500,000	1,410,000	1,200,000	300,000
A-3 + B-2	1,230,000	1,160,000	985,000	245,000

NOTE: See Table 75 for design condition coding summary

As can be seen, the lack of aid for the separate plant at A & P greatly increases the local capital cost, and would cause the local District to lean strongly to the joint plant if it, in turn, is determined eligible. However, it must be noted that, on a capital cost basis, a joint plant would be indicated regardless of aid eligibility.

FORT FAIRFIELD AREA - OPERATING COSTS - Any comparative analysis must consider operational and maintenance costs as well as capital costs, especially as all operating costs must be raised locally. Preliminary estimates of operating costs have been prepared for both a joint plant and a separate plant system. The estimates of operational costs are summarized in Table 113.

TABLE 113

SUMMARY OF OPERATING COSTS

Fort Fairfield

	Separate Plants			Joint Plant	
Industry Loading	A&P $/Yr Condition C-1	Town $/Yr Condition C-3	Total $/Yr Condition C-1 & C-3	$/Yr Condition C-2	Estimated Savings $/Yr
A-1 + B-1	$63,100/yr	$20,500/yr	$83,600/yr	$67,800/yr	$15,800/yr
A-2 + B-1	62,500	20,500	83,000	66,100	16,900
A-3 + B-1	48,000	20,500	68,500	50,100	18,400
A-1 + B-2	60,100	20,500	80,600	63,000	17,600
A-2 + B-2	59,000	20,500	79,500	62,400	17,100
A-3 + B-2	47,000	20,500	67,500	40,000	17,500

NOTE: See Table 75 for design condition coding summary

The costs in Table 113 include labor costs, power costs, chemical costs, heating and general utilities and maintenance. The major items are labor and power costs. Table 113 indicates that, in addition to a capital cost savings, a joint plant will provide an operating cost savings of about $17,000/yr. These savings are mostly in the form of labor costs.

FORT FAIRFIELD AREA - ANNUAL REVENUE REQUIREMENTS - A total cost comparison of systems studied can be achieved by estimating the finance costs for the capital system and combining it with the operating costs to arrive at a total annual revenue requirement for the system. For this analysis, 30 year term bonds at 6% were assumed. A summary of annual finance costs is presented in Table 114, while the total annual costs are summarized in Table 115.

TABLE 114

SUMMARY OF FINANCE COSTS*

Fort Fairfield

Industry Loading	Separate Plants Condition C-1 & C-3	Joint Plant Condition C-2	Savings
A-1 + B-1	$112,800/yr	$23,200/yr	$ 89,600/yr
A-2 + B-1	111,400	22,100	89,300
A-3 + B-1	86,900	18,200	68,700
A-1 + B-2	108,500	22,900	85,600
A-2 + B-2	104,100	21,800	82,300
A-3 + B-2	86,100	17,800	68,300

*Based on Dual Unit System for all configurations
NOTE: See Table 75 for design condition coding summary

TABLE 115

SUMMARY - TOTAL ANNUAL REVENUE REQUIREMENTS

Fort Fairfield

Industry Loading	Separate Plants Condition C-1 & C-3	Joint Plant Condition C-2	Savings
A-1 + B-1	$196,400/yr	$91,000/yr	$105,400/yr
A-2 + B-1	194,400	88,200	106,200
A-3 + B-1	155,400	68,300	87,100
A-1 + B-2	189,100	85,900	103,200
A-2 + B-2	183,600	84,200	99,400
A-3 + B-2	153,600	67,800	85,800

Study of local treatment at Fort Fairfield indicates that the Fort Fairfield Utility District, or other public body, should consider construction of a treatment plant serving both the domestic and industrial wastes in the community. The following advantages are realized with a joint system.

1. Total capital costs of a joint treatment plant are about 20% less than two plants.
2. Governmental aid would not be available for a plant treating only the A & P waste. This has a very large impact on local costs.

3. Operational costs for a joint plant are also about 20% less than for the two plant system.
4. The additional nutrients provided to a joint system by the municipal wastes will be beneficial in operating the biological system.

Future expansion of the treatment plant must also be considered. The foregoing preliminary design and cost estimates are based on a municipal waste load projection to the year 1995. However, the load generated at A & P is assumed to represent present production (with in-plant changes considered). Should the A & P Company greatly increase its production, or should another significant waste producing industry locate in Fort Fairfield, the plant would have to be expanded. While it is practical to project municipal waste loads, it is not practical to anticipate significant industrial expansion unless it is firmly planned. In an industrial treatment facility it is vital to design the initial facility so that it may be expanded with relative ease. This approach has been taken at Fort Fairfield. If it is public policy to provide treatment for A & P and other industry wastes, it will be no more difficult to expand a joint plant at the A & P location than it would a separate plant. This consideration should not preclude a joint effort.

EASTON AREA - CAPITAL COSTS - Cost estimates have been prepared for the local community systems in the Easton area. The facilities estimated include individual systems for each industry and the Town, and for the various combinations outlined in the previous section. The estimates also included the outfall system required to carry the treated wastewaters to the Aroostook River, and the low flow augmentation system required to meet minimum flow conditions in the Prestile. No cost allowances have been made for primary sedimentation facilities at the industry. The loading condition and combination options are coded in the same manner as previously designated in Table 79. The reader is referred to Table 79 for coding definition. The capital cost estimates for the combined industry and industry-domestic systems are presented in Tables 116 and 117.

TABLE 116

CAPITAL COSTS - INDIVIDUAL PLANTS

Easton Area

Vahlsing, Inc Condition A Loading Condition	Est Cost	Maine Sugar Industries Condition B Loading Condition	Est Cost
A-1	$2,400,000	B-1	$1,700,000
A-2	2,100,000	B-2	1,300,000
A-3	1,600,000	B-3	1,000,000

Easton Municipal - Condition C-3 - Est Cost = $420,000

TABLE 116
(Continued)

Transfer Industrial Wastes to Aroostook River - Condition D-1 -
Est Cost = $1,100,000

Total Capital Costs - Ind Systems
Condition C-1

A-1 + B-1 = $5,620,000
A-2 + B-1 = 5,320,000
A-3 + B-1 = 4,820,000

A-1 + B-2 = $5,220,000
A-1 + B-2 = 4,920,000
A-3 + B-2 = 4,420,000

A-1 + B-3 = $4,920,000
A-2 + B-3 = 4,620,000
A-3 + B-3 = 4,120,000

NOTE: See Table 79 for design condition coding summary

TABLE 117

CAPITAL COSTS - JOINT SYSTEMS

Easton Area

Loading Condition	Treatment	Municipal Pumping	Outfall to Aroostook	Total
A-1 + B-2 + C-1*	$3,860,000	0	$1,100,000	$4.970,000
A-2 + B-1 + C-1*	3,760,000	0	1,100,000	4,860,000
A-3 + B-1 + C-1*	3,470,000	0	1,100,000	4,570,000
A-1 + B-2 + C-1*	3,700,000	0	1,100,000	4,800,000
A-2 + B-2 + C-1*	3,350,000	0	1,100,000	3,450,000
A-3 + B-2 + C-1*	2,800,000	0	1,100,000	2,900,000
A-1 + B-3 + C-1*	3,500,000	0	1,100,000	4,600,000
A-2 + B-3 + C-1*	2,900,000	0	1,100,000	4,000,000
A-3 + B-3 + C-1*	2,300,000	0	1,100,000	3,400,000
A-1 + B-1 + C-2	3,890,000	$250,000	1,100,000	5,240,000
A-2 + B-1 + C-2	3,790,000	250,000	1,100,000	5,140,000
A-3 + B-1 + C-2	3,450,000	250,000	1,100,000	4,800,000
A-1 + B-2 + C-2	3,730,000	250,000	1,100,000	5,080,000
A-2 + B-2 + C-2	3,380,000	250,000	1,100,000	4,730,000
A-3 + B-2 + C-2	2,830,000	250,000	1,100,000	4,180,000
A-1 + B-3 + C-2	3,530,000	250,000	1,100,000	4,880,000
A-2 + B-3 + C-2	2,930,000	250,000	1,100,000	4,280,000
A-3 + B-3 + C-2	2,330,000	250,000	1,100,000	3,680,000

*C-1 Conditions include only the cost of industrial waste treatment. For a separate Easton municipal plant, add $420,000.

NOTE: See Table 79 for design condition coding summary.

In Table 117 the C-1 conditions assume the Easton domestic wastes are not included in the joint plant, while the C-2 conditions assume they are included. While the C-1 condition is theoretically possible, it is quite likely that if a joint system is adopted in Easton, it will be accomplished by a public body and the Easton domestic wastes would be included. Therefore, only the C-2 conditions at Easton were used for additional study. Analysis of Tables 116 and 117 indicate that in general, savings can be achieved by adopting a joint industry-municipal system in Easton. These savings are summarized in Table 118.

TABLE 118

SUMMARY - CAPITAL COST SAVINGS - JOINT SYSTEM

Easton Area

Loading Condition	Est Capital Savings
A-1 + B-1 + C-2	$380,000
A-2 + B-1 + C-2	180,000
A-3 + B-1 + C-2	20,000
A-1 + B-2 + C-2	140,000
A-2 + B-2 + C-2	190,000
A-3 + B-2 + C-2	240,000
A-1 + B-3 + C-2	40,000
A-2 + B-3 + C-2	350,000
A-3 + B-3 + C-2	440,000

NOTE: See Table 79 for design condition coding summary

In general, savings of $200,000 to $400,000 can be expected if a joint plant system is adopted for the Easton area. The variations in the figures in Table 118 are to be expected with preliminary estimating procedures. The total savings in capital cost for a joint plant at Easton is not as large as similar facilities in other communities. The savings are tempered by the costs of pumping the municipal wastewater to Easton Station and the cost of transferring the effluent to the Aroostook River.

As presented in the previous section, the installation of advanced waste treatment facilities at Easton is a possible option, in lieu of transfer to the Aroostook River. While a detailed cost analysis has not been conducted, a rough analysis suggests cost ranges for comparison with transfer. Stephan, etal[40] reported on the status of advanced treatment and wastewater renovation in 1970, giving approximate cost values. The approximate cost of full advanced wastewater treatment facilities at Easton, based on the cited published data, is presented in Table 119.

TABLE 119

APPROXIMATE CAPITAL COSTS - ADVANCED WASTE TREATMENT

CONDITION D-3

Easton Area

Loading Condition	Approx Capital Cost
A-1 + B-1	$8,000,000
A-2 + B-1	7,200,000
A-3 + B-1	5,800,000

NOTE: See Table 79 for design condition coding summary

Although the above table represents an approximate cost only, it is evident that the installation of full advanced wastes treatment facilities at Easton is much more costly than transfer to the Aroostook River, assuming such transfer can be legally accomplished. This is discussed in detail in the companion Basin Planning Report.

The local share of the Easton system cost will be lessened by application of State and Federal aid programs as described in the introduction to this section. The effect of this aid is illustrated in Tables 120 and 121.

TABLE 120

LOCAL CAPITAL COSTS

Easton Area

Individual Systems

Vahlsing, Inc Condition A	Maine Sugar, Ind. Condition B	Transfer Condition D-1	Easton Condition C-3	Total
A-1 = $2,400,000	B-1 = $1,700,000	$1,100,000	$61,000	$5,261,000
A-2 = 2,100,000	B-2 = 1,300,000	1,100,000	61,000	4,561,000
A-3 = 1,600,000	B-3 = 1,100,000	1,100,000	61,000	3,761,000

NOTE: Est total costs for other combination of loadings can be found by alternating the A & B load combinations, such as A-1, B-2, etc. See Table 79 for design condition coding summary.

TABLE 121

LOCAL CAPITAL COSTS

Easton Area

Joint Municipal-Industry System
Condition C-2 & D-1

Loading Condition	Est Total Local Cost	Typical Savings Joint System
A-1 + B-1 + C-2	$790,000	$4,471,000
A-2 + B-1 + C-2	780,000	
A-3 + B-1 + C-2	730,000	
A-1 + B-2 + C-2	770,000	
A-2 + B-2 + C-2	710,000	3,851,000
A-3 + B-2 + C-2	630,000	
A-1 + B-3 + C-2	740,000	
A-2 + B-3 + C-2	650,000	
A-3 + B-3 + C-2	560,000	3,201,000

Analysis of Tables 120 and 121 indicate significant local capital cost savings if a joint system is undertaken by a public body. These savings will range from 3 million to 4 million dollars, depending on the loading combinations finally adopted. These savings are largely realized by application of the grant-in-aid factors, although a joint system would be the most economical regardless of aid factors.

EASTON AREA - OPERATING COSTS - Estimates have been prepared for the operation costs of both the individual and combined plants in the Easton area. These costs include labor, power, chlorine and other chemicals, and general operation and maintenance. These costs assume a 10 month operation at both Vahlsing, Inc and Maine Sugar Industries. It is recognized that the beet processing campaign would only last 100 to 130 days. However, plans have been proposed for processing imported cane during other periods. If only beets are processed, the operating costs could be reduced accordingly. Estimated operating costs are presented in Table 122.

TABLE 122

SUMMARY OPERATING COSTS

Easton Area

Individual Systems
Conditions A, B, & C-3

Vahlsing	Est Cost	Maine Sugar Ind	Est Cost	Laston Cost	Total Cost
A-1	$124,300/yr	B-1	$110,000/yr	$17,000/yr	$251,300/yr
A-2	105,200	B-2	80,500	17,000	202,700
A-3	87,000	B-3	67,500	17,000	171,500

NOTE: Est total operating costs for other combinations of loadings can be found by altering the A & B load conditions, such as A-1, B-2, etc.

Joint System
Conditions C-2 & D-1

Loading Condition	Est Oper Cost	Typical Savings $/Yr Joint System
A-1 + B-1 + C-2	$232,000/yr	$18,700/yr
A-2 + B-1 + C-2	214,000	
A-3 + B-1 + C-2	193,000	
A-1 + B-2 + C-2	218,000	
A-2 + B-2 + C-2	191,000	$11,700/yr
A-3 + B-2 + C-2	166,000	
A-1 + B-3 + C-2	195,000	
A-2 + B-3 + C-2	175,000	
A-3 + B-3 + C-2	154,000	17,500/yr

NOTE: See Table 79 for design condition coding summary

Analysis of Table 122 indicates that a joint treatment system in Easton, receiving both the industrial and municipal wastes, has significant advantages over individual treatment facilities.

EASTON AREA - ANNUAL REVENUE REQUIREMENTS - The total annual costs required to support the Easton area facilities consist of the operating costs, as outlined above, plus the finance costs to support the local bonds. Again, 30 year bonds at 6% has been assumed for preliminary design analysis. It is recognized that industry would utilize other financing procedures if individual plants were adopted, but the data presented provides reasonable comparative data. Table 123 summarizes the finance costs, while Table 124 summarizes the total annual revenue requirements.

TABLE 123

SUMMARY - SYSTEM FINANCE COSTS

Easton Area

Individual Systems

Vahlsing Condition A	Maine Sugar Ind Condition B	Transfer Condition D-1	Easton Condition C-3	Total
A-1 = $174,000/yr	B-1 = $124,000/yr	$80,000/yr	$4,500/yr	$382,500/yr
A-2 = 153,000	B-2 = 95,000	80,000	4,500	332,500
A-3 = 116,000	B-3 = 73,000	80,000	4,500	273,500

Joint Systems
Conditions C-2 & D-1

Loading Conditions	Est Cost	Typical Cost Savings
A-1 + B-1 + C-2	$57,400/yr	$325,100/yr
A-2 + B-1 + C-2	56,600	
A-3 + B-1 + C-2	53,100	
A-1 + B-2 + C-2	56,000	
A-2 + B-2 + C-2	51,600	280,900
A-3 + B-2 + C-2	46,700	
A-1 + B-3 + C-2	53,700	
A-2 + B-3 + C-2	47,200	
A-3 + B-3 + C-2	40,600	232,900

NOTE: See Table 79 for design condition coding summary

TABLE 124

SUMMARY - ANNUAL REVENUE REQUIREMENTS

Easton Area

Individual Systems

Vahlsing Condition A	Maine Sugar Ind Condition B	Transfer Condition D-1	Easton Condition C-3	Total
A-1 = $298,300/yr	B-1 = $234,000/yr	$80,000/yr	$21,500/yr	$633,800/yr
A-2 = 258,200	B-2 = 175,500	80,000	21,500	535,200
A-3 = 203,000	B-3 = 140,500	80,000	21,500	445,000

Joint Systems
Conditions C-2 & D-1

Loading Conditions	Est Annual Cost	Typical Cost Savings
A-1 + B-1 + C-2	$289,400/yr	$343,400/yr
A-2 + B-1 + C-2	270,600	
A-3 + B-1 + C-2	246,100	
A-1 + B-2 + C-2	274,000	
A-2 + B-2 + C-2	242,600	292,600
A-3 + B-2 + C-2	212,700	
A-1 + B-3 + C-2	248,600	
A-2 + B-3 + C-2	222,200	
A-3 + B-3 + C-2	194,600	250,400

NOTE: See Table 79 for design condition coding summary

Analysis of Table 124 indicates a significant overall local savings by adoption of a joint plant system for Easton. Much of this savings is in the finance costs, which reflect both the overall capital cost reductions by a joint plant and aid eligibility for public systems.

The costs presented in the foregoing paragraphs do not reflect the cost of pumping water from the Aroostook River, for low flow augmentation of the Prestile during drought conditions. The estimated capital cost for these facilities is $520,000. These costs must be incurred regardless of whether joint or individual systems are adopted in Easton. If individual systems are adopted, the low flow augmentation facilities would not be eligible for State and Federal aid. If they were constructed as part of a general public system, their aid eligibility is uncertain. However, it is believed that a sound case can be made for their inclusion, as the alternatives would be somewhat more costly to all levels of government. If eligible for aid, the local finance costs would be estimated at about $6,000/yr and operating costs could run from $2,000 to $6,000/yr depending on required usage. These costs would be additive to those presented in the foregoing paragraphs.

Study of local treatment at Easton indicates that the Town, or other public body, and local industries should consider joint treatment of domestic and industrial wastes in the community. The potential economic advantages to all parties far outweigh the administrative and financial negotiation problems inherent in such a system. The following are advantages of a joint system:

1. Total capital costs of a joint treatment plant are about 5% less than multiple plants, if equally flexible and reliable systems are installed.
2. Governmental aid would not be available for plants treating only the industrial wastes. This has a very large impact on local costs.
3. Operational costs for a joint plant are also about 6 to 10% less than for the multiple plant system.
4. The additional nutrients provided to a joint system by the municipal wastes will be beneficial in operating the biological system.

Future expansion of the treatment plant must also be considered. The foregoing preliminary design and cost estimates are based on a municipal waste load projection to the year 1995. The loads generated at local industries are assumed to represent present production (with in-plant changes considered). Should the industries greatly increase their production beyond those projected, or should another significant waste producing industry locate in Easton, the plant would have to be expanded. While it is practical to project municipal waste loads, it is not practical to anticipate significant industrial expansion unless it is firmly planned. In an industrial treatment facility, however, it is vital to design the initial facility so that it may be expanded with relative ease. This approach has been taken at Easton. If it becomes public policy to provide treatment of industry wastes, it will be no more difficult, and probably less so, to expand a joint plant than it would a separate plant.

REGIONAL SYSTEMS

WASHBURN - CORE AREA INTERCONNECT - As described under the preliminary design sections, facilities were studied to interconnect the Washburn area to the Core Area at Presque Isle. An economic analysis of these facilities has been made, including evaluation of capital and operating costs.

The Washburn-Core Area interconnect was designed as both a gravity flow and a pumped system. The estimated capital costs for the physical interconnect systems are:

Gravity System	=	$2,902,000
Pumped System	=	2,480,000
Difference	=	$ 442,000

It is evident that the pumped system is somewhat more economic than the gravity system due to the difficult construction conditions that would be encountered along the river.

In addition to the capital costs of the interconnect system, the cost of treatment must be considered. Primary sedimentation for solids removal at Washburn will be required prior to entry into the interconnect system. These facilities are estimated at $280,000 to $300,000 depending on the Taterstate loading condition.

It will also be necessary to treat the Washburn area wastewaters in the Core Area plant after transfer. Studies of a regional treatment facility suggest that the wastewaters can be treated in a large regional facility for about $15 per population equivalent. If this factor is applied to the Washburn wastewaters, the Washburn component of regional treatment costs would range from about $800,000 to $1,400,000, depending on the Taterstate load condition. With these treatment costs included, an approximate capital cost comparison between an interconnected and local system was prepared. This comparison is presented in Table 125.

TABLE 125

INTERCONNECTED VERSUS COMMUNITY SYSTEM*

Washburn Area - Condition C-2

Unit	Cost-Condition A-1+B-2	Cost-Condition A-2+B-2	Cost-Condtion A-3+B-2
Local Treatment	$2,314,000	$2,174,000	$1,737,000
Regional System			
Transport Costs	2,902,000	2,850,000	2,800,000
Primary Treatment Costs	300,000	290,000	280,000
Regional Treatment Costs	1,400,000	1,100,000	800,000
Total Regional System	$4,602,000	$4,240,000	$3,880,000
Est Savings Local System	$2,288,000	$2,066,000	$2,143,000

*Costs based on pumped interconnect system.
NOTE: See Table 66 for design condition coding summary.

From Table 125 it is evident that an interconnected system between Washburn and the Core Area will cost significantly more than a local system. Even if Federal and State aid factors were applied, the added local cost would amount to over $500,000. An approximation of comparative operating costs has also been prepared. These costs are summarized in Table 126.

TABLE 126

COMPARATIVE OPERATING COSTS - INTERCONNECT VS LOCAL

Washburn Area - Condition C-2

	Taterstate Design Condition		
Cost Unit	A-1+B-2	A-2+B-2	A-3+B-2
Primary Plant-Taterstate	$18,000/yr	$17,000/yr	$15,000/yr
In Route Transport	13,500	13,000	12,000
Pumping at Presque Isle	4,000	3,800	3,000
Regional Treatment	63,000	57,200	44,000
Totals	$98,500/yr	$91,000/yr	$74,000/yr
Est Oper Costs-Local System	$82,800/yr	$76,800/yr	$60,300/yr
Savings for Local System	$15,700/yr	$14,200/yr	$13,700/yr

NOTE: See Table 66 for design condition coding summary

It is estimated that an operating cost savings of $14,000 to $16,000 per year can be achieved by adoption of the local Washburn system. To evaluate both systems on a total annual cost basis, the average annual finance cost of the additional $2,000,000± capital cost was computed using 30 year term bonds at 6% interest. This average annual finance cost is estimated at $145,000 per year. If based on only local cost, the additional annual finance cost is estimated at $40,000 per year. With the total capital cost increase considered, the transfer of the Washubrn wastewaters to the Core Area for treatment will cost an estimated additional $160,000 per year. If only local financing is considered, the additional annual local cost is estimated at $54,000 per year.

To complete the analysis of the feasibility of transferring the Washburn wastewaters to the Core Area for treatment, other, non-direct cost elements have been considered. These elements are:

1. Comparative Service Area
2. Flexibility for Growth at Washburn
3. Effect on Water Quality
4. General Operational Difficulties

The interconnection of the Washburn system with the Core Area will not increase the basic service area provided by the system over that provided by treatment at Washburn if the lower cost pressurized system is used. If the gravity interceptor along the river is used, the basic service area is expanded greatly. The expanded area would include Crouseville Village, and all tributary drainage area between Washburn and Presque Isle. Most of this additional service area is currently in agricultural production, and is likely to stay in this usage for the foreseeable future. It must be recognized that the added service area is only of value if extensive development is goint to take place in a reasonable period of time. The anticipated growth rate in the Washburn area is such that only a small portion of the added service area will actually be used during a 20 year design period.

The interconnected system will not provide the Town of Washubrn with the flexibility for growth that a local treatment system will. The major cost of interconnection is the force main, or gravity interceptor, connecting Washburn and the Core Area. This must be sized, at this time, for a reasonably anticipated flow. If too large, the time of transfer becomes so great that operating and maintenance problems would be expected. Also, such oversizing at this time would be speculative in anticipation of significant growth, and would tie up capital funds which may be needed for other community projects. Once installed, the only way to gain any significant increase in capacity is to build a parallel line; whereas, a local treatment plant at Washburn can be designed for ready expansion as the demand arises. Thus, the physical interconnection could prove to be a constraint on growth in the Washburn area.

The water quality standards established for the Aroostook River can be met by either a local treatment plant or the interconnected regional system. Discharge of treated wastewater at Washburn will affect the water quality below the discharge. The limiting factor on discharge at Washburn is the B-1 classification of the river in the vicinity of Caribou. The treatment proposed at Washburn, however, would meet this classification with reasonable allowance for growth in the area. Studies indicate the oxygen sag point in the river created by organic discharge at Washburn would be located between Presque Isle and Caribou.

Carrying the Washburn wastewater to the Core Area has the potential advantage of discharge below the Caribou water intake. If this were accomplished, essentially all significant waste discharges above Caribou would be eliminated. This would have the effect of tending to lower water quality in the Aroostook River below Caribou. More detailed discussions of waste load impact on the river is presented in the companion Basin Planning Report. On balance, however, no great gain in river water quality would be obtained by connecting Washburn to the Core Area.

The piped interconnecting system would operate satisfactorily while the Taterstate plant is in operation. However, during periods when the Taterstate plant is not operating, the domestic wastewaters must still be transferred by the pipeline. With flows less than 30% of design, the transfer time would increase significantly and velocities would be lowered somewhat. This may cause operating difficulties in keeping the flow fresh. While perhaps not critical, such conditions must be considered a disadvantage for interconnection.

Based on the study of potential benefits which may accrue from regional interconnection of the Washburn wastewaters with the Core Area system, it must be concluded that they do not warrant the significant additional capital and operating costs of such interconnection. The wastewaters generated at Washburn should be treated locally prior to discharge to the Aroostook River.

MAPLETON - CORE AREA INTERCONNECT - As described under the preliminary design section, facilities were studied to interconnect the Mapleton area with the Core Area at Presque Isle. An economic analysis of these facilities has been made for comparison purposes.

The Mapleton-Core Area interconnect was designed on both gravity and pumped systems. The estimated capital costs for the physical interconnect systems are:

Gravity System = $1,062,000
Pumped System = $ 935,000
Difference = $ 127,000

In addition to the interconnection costs, additional capital costs must be allowed for treatment capacity in the regional system. This, however, will be quite small due to the large economies of scale. The capital cost increase to the regional treatment system to handle the Mapleton wastes would not exceed $15 per capita, or about $15,000. Also, as the flows are quite small, at least initially, it has been assumed that the Mapleton flow could pass through the Presque Isle sewer system with no increase in size required. While this would have to be checked in detail prior to construction, it is a reasonable assumption for comparative purposes.

With the treatment cost included, the total capital cost of handling the Mapleton wastewater through the regional system is:

Stream Route	$1,077,000
Route 163 Route	$ 950,000
Difference	$ 127,000

With Federal and State aid applied at the 85% factor, the local costs of the interconnected system are:

Stream Route	$ 160,000
Route 163 Route	$ 142,500
Difference	$ 17,500

The capital costs of the interconnected system can be compared with the capital cost of locally treating the Mapleton wastewaters. Capital cost estimates for the Mapleton treatment plant are estimated at $320,000.

From a capital cost standpoint, the local treatment system is somewhat more economic than physical interconnection. This is summarized in Table 127. This table assumes use of the less costly Route 163 interconnection route.

TABLE 127

COST COMPARISON-INTERCONNECTION VERSUS LOCAL TREATMENT

Mapleton Area

Unit	Total Capital Cost	Local Capital Cost
Interconnected System	$950,000	$143,000
Local System	320,000	48,000
Savings, Local System	$630,000	$ 95,000

In addition to capital costs, the systems have been compared on an operating cost basis. The estimated system operating costs for both systems is $12,000/yr to $15,000/yr. Thus, from a cost standpoint it will be somewhat more costly to interconnect the Mapleton wastewaters to the Core Area at Presque Isle.

Again, it must be recognized that factors other than direct costs must be considered when evaluating local and regional pollution control programs. These factors include consideration of the following factors:

1. Comparative service area and need for service.
2. Effect on water quality and downstream water uses.
3. Effect on economic growth and development of the Town and the Region.

The interconnected regional system will significantly increase the potential service area. The interconnection route along the Presque Isle Stream would provide the basic service for eventual development of the Presque Isle Stream watershed between Mapleton and Presque Isle. However, the land adjacent to this route is currently undeveloped and, in many areas, low, swampy land will preclude development. The interconnection route along Route 163 would provide additional service area also, and would provide immediate service to about 40% of the length of Route

163 between Mapleton and Presque Isle. Additional gravity sewers not included in the estimates of this report would be required to service the remianing 60%. Strip development along Route 163 would be encouraged unless the Town takes steps to discourage such development.

The primary demand for development will probably be in the vicinity of Route 163 near Mapleton Village. The areas along Route 163, as it approaches Presque Isle, tend to be low and wet and are only marginal for development. It is estimated that the section of Route 163, and adjacent areas, from the point where it crosses the Presque Isle Stream to Mapleton Village, could be provided with basic sewer service through the local Mapleton system, equivalent to that provided by the regional system, at a cost of about $278,000. If this cost were added to the treatment cost at Mapleton, the total capital cost would amount to $578,000. This can be compared to $955,000 for the fully interconnected regional system. Thus, the local service could be expanded to include most of the desirable land for development in Mapleton at somewhat less capital cost than through full interconnection.

The demand for added service area must also be considered. The population of Aroostook County as a whole has been declining, with a decrease of 12.8% in the last 10 years. However, the Town of Mapleton has increased in population by 5.5% during this same period. With its close proximity to the "Core Area" of Presque Isle and Caribou, a modest continued residential growth in Mapleton will probably continue. However, the need for service in the vast areas possible by regional interconnection cannot possibly develop within a 20 year planning period, although some service adjacent to Route 163 near Mapleton Village may well be warranted.

There are two primary considerations when evaluating the effect of treated effluent discharges at Mapleton on water quality. These are: the water quality at the Presque Isle Water District intake, and the possible multi-purpose impoundment on the North Branch of the Presque Isle Stream downstream of Mapleton.

The current classification of the North Branch of the Presque Isle Stream below Mapleton is B-2. The discharge from the proposed Mapleton treatment plant will not lower this classification. If properly treated, the discharge from the Mapleton treatment plant will not have any adverse effect on the water quality provided by the Presque Isle Water District treatment plant, nor will it have any effect on the operating cost of the Presque Isle plant. Thus, while desirable to eliminate any effluent discharge above the water intake, the value gained by removing the Mapleton discharge is small in comparison to the cost of doing so.

As part of its Presque Isle Stream watershed plan, the Soil Conservation Service has proposed a dual-purpose impoundment on the Lower North Branch of the Presque Isle Stream below Mapleton. The structure will be for flood control, and will provide a permanent pool for municipal recreational use. The Town of Mapleton has undertaken considerable planning for the recreational aspects of this impoundment. To date, the recreational development at this site is uncertain. The shallow depths of the permanent pool, together with the high organic content of the underlying soil, creates doubt concerning the water quality of the pool for recreational purposes. Concern has also been expressed on the discharge of treated effluent from the Mapleton plant above this location.

The discharge from the Mapleton treatment plant should in no way create any health hazard if recreational use is made of the permanent pool of the Lower North Branch impoundment. It is recognized that the plant effluent will contribute to the nutrient input to the pool. However, this input will be small in comparison to that from natural runoff from forest and agricultural lands in the area. Also, considerable amounts of nutrients are readily available in the organic peat which covers most of the proposed impoundment area. The significance of municipal effluent, therefore, in the overall water quality of the permanent pool is questionable.

For preliminary evaluation, nutrient (phosphorous) removal at the Mapleton plant was considered. Moderate reductions could be achieved by addition of alum to the aeration system and utilizing the surplus capacity available from the solids dewatering unit to handle the increased solids removal required by such a procedure. This removal technique would require little additional capital cost, perhaps $10,000 to $15,000. If higher removals were required, a separate coagulation, sedimentation, and filtration unit could be provided at an added cost of about $80,000. Thus, even if phosphorous removal was warranted, the costs would be somewhat lower than regional interconnection.

Industrial and economic development is a prime concern to communities in northern Maine. The Town of Mapleton cannot effectively attract wet process industries with today's concern for environmental problems. The capacity of the Presque Isle Stream to receive waste loads is definitely limited and, while reasonable population growth can be easily accommodated by a local treatment system, large industrial loads definitely cannot. This would tend to suggest a regionally interconnected system. Installation of such a system at this time, however, to handle a wet process industry would be very speculative and tie up large amounts of local capital. If the regional system was installed as outlined herein, definite limits on capacity exist. Any wet process industry locating in Mapleton would have to connect to the Presque Isle, or regional system. This will prove a definite economic disadvantage and will discourage such industry. The development efforts in Mapleton, therefore, should be directed in the "Dry Process" area, with such development being within the capacity of the local system. With all factors considered, it is very doubtful if interconnection of the Mapleton wastewaters to the Core Area can be justified.

FORT FAIRFIELD - CORE AREA INTERCONNECT - As described in the preliminary design section, facilities were studied to interconnect the Fort Fairfield area with the Core Area at Grimes Mill. An economic analysis of these facilities has been prepared, including evaluation of capital and operating costs. The Fort Fairfield interconnect was studied in both directions, ie, the Fort Fairfield area wastes carried to Grimes Mill, and the Core Area wastes carried to Fort Fairfield for treatment. The capital costs of the physical interconnections are:

Fort Fairfield to Grimes Mills = $1,725,000
Core Area to Fort Fairfield = $3,200,000

In addition to the costs of the physical interconnect system, the cost of treatment in the regional system must be considered. The A & P wastewaters must receive primary treatment prior to entering the interconnect system. These facilities are estimated at $150,000 to $200,000, depending on load conditions at A & P.

It will be necessary to treat the A & P wastewaters in the Core Area biological plant after transfer. This treatment can be accomplished at an approximate cost of $15 per population equivalent. If this factor is applied to the Fort Fairfield wastewaters, the Fort Fairfield component of regional treatment costs would be about $600,000 to $900,000 depending on A & P load conditions. With these treatment costs included, an approximate capital cost comparison for the Fort Fairfield to Grimes Mill interconnect, and a local community system can be undertaken. This comparison is presented in Table 128.

TABLE 128

INTERCONNECTED VERSUS COMMUNITY SYSTEM

Fort Fairfield to Grimes Mill - Condition C-2

Unit	Cost Condition A-1+B-2	Cost Condition A-2+B-2	Cost Condition A-3+B-2
Local Treatment	$1,480,000	$1,440,000	$1,170,000
Regional System			
Transport Cost	$1,725,000	$1,650,000	$1,500,000
Primary Treatment Cost	200,000	175,000	150,000
Regional Treatment Cost	900,000	800,000	600,000
Est Savings - Local System	$1,345,000	$1,185,000	$1,080,000

NOTE: See Table 75 for design condition coding summary

From Table 128 it is apparent that a Fort Fairfield-Grimes interconnect will be nearly double the cost of the local system. Even if Federal and State aid factors are applied to, the added cost will be significant in light of the overall project cost.

An approximate operating cost analysis of the Fort Fairfield-Grimes interconnect, similar to that reported in detail under the Washburn discussion, indicates approximately equal operating costs between the regional and local system, with possibly some slight saving for the regional system.

To complete the analysis of the feasibility of transfer of Fort Fairfield wastewaters to Grimes Mill for treatment other, non-direct cost elements have been considered. These elements are:

1. Comparative Service Area
2. Flexibility for Growth at Fort Fairfield
3. Effect on Water Quality
4. General Operational Difficulties.

Carrying the Fort Fairfield water to Grimes Mill will not increase the basic service area over that provided by treatment at Fort Fairfield. This is due to the pressurized system interconnecting Fort Fairfield and Grimes Mill.

The interconnected system will not provide the Town of Fort Fairfield with the flexibility for growth that a local treatment system will. The major cost of interconnection is the force main connecting Fort Fairfield and Grimes Mill. This must be sized, at this time, for a reasonably anticipated flow. If too large, the time of transfer becomes so great that operating and treatment problems can be expected. Once installed, the only way to gain any significant increase in capacity is to build a parallel line; whereas, a treatment plant at Fort Fairfield can be designed for ready expansion as the demand arises. The physical interconnection would prove to be a constriant on industrial expansion at Fort Fairfield.

The water quality standards established for the Aroostook River can be met by either system. No advantage is gained by transferring the Fort Fairfield wastewaters to Grimes Mill.

The piped system will operate satisfactorily while the A & P plant is in operation. During periods when the A & P plant is not operating, however, the domestic wastewaters still must be transferred by the pipeline. With flows less than 50% of design, the transfer time will be increased significantly. This again may cause operating difficulties in keeping the flow fresh.

It is not desirable or economically justified to interconnect the Fort Fairfield system with regional system by pumping to Grimes Mill.

A second analysis was made of physical interconnection by carrying the regional wastewaters to Fort Fairfield for treatment. The capital cost of this interconnection is estimated at $3,200,000. If this were accomplished, the incremental Fort Fairfield cost of treating the Fort Fairfield wastes in the regional system would be $600,000 to $900,000. As the size of a regional plant would not greatly increase by addition of the Fort Fairfield load, no unit reduction in treating the regional wastes would be anticipated. Thus the net additive cost for such transfer is estimated at about $2,600,000. Operational cost differentials would be similar to those outlined in the previous section.

The total added capital cost can only be justified if significant added non-direct benefits can be gained. The four items listed previously have been reviewed in light of the large scale regional interconnection.

The gravity interconnection will provide a large increase in potential service area. However, it must be recognized that this is only of value if entensive development is going to take place within a reasonable period of time. While growth in Fort Fairfield will, in all likelihood, occur, only a small portion of this added service area will actually be used. Study of the topography of the added service area indicates that 60% to 70% of it is in the Hockenhull Brook watershed, which drains to a point within one mile of the Fort Fairfield treatment plant site. Thus, 60% to 70% of the added service area could be also served by the Fort Fairfield local system with relatively little added cost. As the service area in the Hockenhull watershed is more than sufficient to meet any reasonable needs of the Town, the value of the remaining service area added by the regional interconnection is relatively small.

The flexibility for growth in Fort Fairfield is essentially the same for both systems as long as the treatment facility is located in Fort Fairfield. However, the connecting pipeline could impose some restraint on upstream industrial growth.

The extension of the regional system downstream to Fort Fairfield would tend to upgrade the water quality of the Aroostook River between Grimes Mill and Fort Fairfield, and would tend to lower it below Fort Fairfield.

In both cases, however, the water quality standards of the river could be met with proper treatment. Even with the transfer of wastewaters to Fort Fairfield, it is doubtful if the Aroostook River below Caribou could be raised to a "B" classification, especially as long as the Caribou municipal sewer system is combined in nature. The urban runoff from Caribou will likely cause frequent coliform bacteria counts above the "B" classification limit. No significant water quality benefits can be gained by bringing the Core Area wastes to Fort Fairfield for treatment.

In summary, it must be concluded that the physical interconnection of Fort Fairfield with the Core Area regional system is not a valid concept, and all Fort Fairfield wastewaters should be treated locally.

CORE AREA INTERCONNECTIONS - As described under the preliminary design section, two basic Core Area interconnections were considered; namely, interconnection of the Easton area wastes with those of the Presque Isle area for joint treatment, and separate treatment of the Caribou wastes; and full interconnection of all Core Area wastewaters. These alternatives represent a two plant, and a single plant system, respectively. Cost analyses have been undertaken for each of these alternatives. Federal aid and other cost factors are the same as those described in earlier paragraphs of this section. Cost estimates are coded as described in earlier sections. The reader is referred to those sections for coding definitions.

The estimated capital costs of the Easton-Presque Isle joint system are presented in Table 129.

TABLE 129

SUMMARY CAPITAL COSTS - JOINT PRESQUE ISLE-EASTON

Loading Condition	Est Capital Cost
IV - a	$10,075,00
IV - c	8,800,000
IV - g	8,910,000
IV - i	7,940,000
VI - a	7,560,000
VI - c	6,800,000
VI - g	6,910,000
VI - i	6,140,000

NOTE: See Table 83 for design condition coding summary

State and Federal aid has been applied to the combined Easton-Presque Isle system, in a similar manner to the individual systems outlined in the foregoing paragraphs. A summary of the estimated local cost of the joint Easton-Presque Isle system is presented in Table 130.

TABLE 130

SUMMARY - LOCAL CAPITAL COSTS

JOINT PRESQUE ISLE - EASTON SYSTEM

Loading Condition	Est Local Cap Costs
IV - a	$1,510,000
IV - c	1,320,000
IV - g	1,340,000
IV - i	1,200,000
IV - a	1,130,000
VI - c	1,040,000
VI - g	1,040,000
VI - i	920,000

NOTE: See Table 83 for design condition coding summary

A comparison of the total capital costs and local capital costs of the joint system with those of individual community systems is presented with appropriate discussion in the next section.

Operating costs have also been estimated for the joint Presque Isle-Easton system. These include labor, power, chlorine and other chemicals, and other operating costs in a similar manner to the local community systems. A summary of the estimated operating costs is presented in Table 131. These are compared with the local system operating costs in the next section.

TABLE 131

SUMMARY OPERATING COSTS

JOINT PRESQUE ISLE - EASTON SYSTEM

Loading Condition	Est Operating Cost
IV - a	$451,000/yr
IV - c	403,000
IV - g	404,000
IV - i	362,000
VI - a	344,000
VI - c	319,000
VI - g	320,000
VI - i	295,000

NOTE: See Table 83 for design condition coding summary.

The total operating costs of Table 131, plus the finance costs to support the local capital costs represent the annual revenue requirements necessary to support the joint Presque Isle-Easton system. Comparable to the preceding analyses, 30 year term bonds at 6% were assumed. The total annual revenue requirements are presented in Table 132.

TABLE 132

SUMMARY - ANNUAL REVENUE REQUIREMENTS

JOINT PRESQUE ISLE - EASTON SYSTEM

Loading Condition	Est Annual Rev Req
IV - a	$560,000/yr
IV - c	499,000
IV - g	500,000
IV - i	450,000
VI - a	426,000
VI - c	395,000
VI - g	396,000
VI - i	362,000

NOTE: See Table 83 for design condition coding summary

The annual revenue requirements presented in Table 132 do not include the costs of low flow augmentation to maintain minimum Prestile Stream flows.

As discussed under the local Easton system, such facilities may be required to meet the legal requirements of interbasin transfer. As the aid factors for such facilities are uncertain, the costs are reported separately. The capital costs are estimated at $520,000, and the operating costs will run $4,000 to $6,000/yr depending on the required usage.

The second alternative for Core Area interconnection is carrying the Easton and Presque Isle area wastewaters downstream to Caribou for treatment. As described under the preliminary design section, two interconnecting concepts were studied; namely, the standard piped interconnect system and the transport-treatment concept. A cost analysis of the two full interconnect systems have been prepared. As presented earlier, three loading conditions were studied. The final treatment plant of this interconnect system would be located at Grimes Mill. As discussed under the design section, the in-town site is not adequate for treatment of the entire Core Area load. The estimated capital costs of the standard piped interconnect is presented in Table 133.

Table 133

SUMMARY - CAPITAL COSTS

PIPED INTERCONNECT SYSTEM - CORE AREA

Unit	Cap Cost Max Load	Cap Cost Med Load	Cap Cost Min Load
PQI to PSI Transport	$ 450,000	$ 450,000	$ 450,000
Easton to PSI Transport	1,550,000	1,400,000	--
Low Flow Augmentation	520,000	520,000	--
PSI to Caribou Transport	4,600,000	4,200,000	4,000,000
ADJ Caribou In-town Plant	1,100,000	900,000	750,000
In-town to Grimes Transport	1,300,000	1,000,000	900,000
TP at Grimes Site	12,300,000	9,200,000	4,500,000
Totals	$21,820,000	$17,770,000	$10,600,000

State and Federal aid factors would apply to the interconnected system in a similar manner to that discussed under the local system options. The estimated local costs are summarized in Table 134.

TABLE 134

SUMMARY - LOCAL CAPITAL COSTS

PIPED INTERCONNECT SYSTEM - CORE AREA

Loading Condition	Est Local Cap Cost
Maximum	$3,280,000
Medium	2,670,000
Minimum	1,590,000

Operating costs have also been estimated for the standard piped interconnected system. These costs were estimated on the same basis as the individual system options. The estimated operating costs are summarized in Table 135.

TABLE 135

SUMMARY - OPERATING COSTS

PIPED INTERCONNECT SYSTEM - CORE AREA

Loading Condition	Est Oper Cost
Maximum	$750,000/yr
Medium	560,000
Minimum	320,000

The total annual revenue required to support the Core Area interconnected system is the sum of the operating costs as listed in Table 135, plus the finance charges for the local capital costs. The finance charges were computed on 30 year term bond conditions at 6%. The finance costs and the annual revenue requirements are summarized in Table 136.

TABLE 136

FINANCE COSTS AND ANNUAL REVENUE REQUIREMENTS

PIPED INTERCONNECT SYSTEM - CORE AREA

Loading Condition	Finance Cost	Annual Rev Req
Maximum	$238,000/yr	$988,000/yr
Medium	194,000	754,000
Low	116,000	436,000

Similar analyses have been prepared for the transport-treatment method of Core Area interconnection. The capital cost estimate for this system is presented in Table 137.

TABLE 137

SUMMARY - CAPITAL COSTS

TREATMENT-TRANSPORT SYSTEM - CORE AREA

Unit	Cap Cost Max Load	Cap Cost Med Load	Cap Cost Min Load
PQI to PSI Transport	$ 450,000	$ 450,000	$ 950,000
Easton to PSI Transport	1,550,000	1,400,000	--
Low Flow Augmentation	520,000	520,000	--
Treatment-Transport Channel	11,500,000	10,500,000	9,000,000
Subtotal-System to Caribou Plant	$14,020,000	$12,870,000	$9,450,000
TERMINAL FACILITIES			
Option 1 - In-town Caribou	8,085,000	7,000,000	4,000,000
Total W/Option 1	$22,105,000	$19,870,000	$13,450,000
Option 2 - In-town Caribou	7,685,000	6,290,000	3,700,000
Total W/Option 2	$21,705,000	$19,160,000	$13,150,000
Option 3 - Grimes Site	8,635,000	6,090,000	5,315,000
Total W/Option 3	$22,655,000	$19,960,000	$14,765,000

State and Federal aid factors would apply to the treatment-transport concept, in a similar manner to the piped system. The local costs of the system are summarized in Table 138.

TABLE 138

SUMMARY - LOCAL CAPITAL COSTS

TREATMENT-TRANSPORT SYSTEM - CORE AREA

Terminal Option	Local Cost Max Load	Local Cost Med Load	Local Cost Min Load
Option 1	$3,310,000	$2,980,000	$2,020,000
Option 2	3,260,000	2,870,000	1,970,000
Option 3	3,400,000	2,990,000	2,210,000

Operating costs have also been estimated for the treatment-transport system. These costs are similar to, but slightly higher than the piped system. This is due to the travel required to service the channel system and aeration units plus a lower oxygen transfer efficiency. The estimated operating costs are summarized in Table 139.

TABLE 139

SUMMARY - OPERATING COSTS

TREATMENT-TRANSPORT SYSTEM - CORE AREA

Loading Condition	Est Oper Costs
Maximum	$780,000/yr
Medium	610,000
Minimum	370,000

The total annual revenue required to support the Core Area treatment-transport system is in the sum of the operating costs, as listed in Table 139, plus the finance charges for the local capital cost. The finance charges were computed for 30 year term bonds at 6%. The finance costs and the annual revenue requirements are presented in Table 140, assuming the Grimes site is adopted as the terminal plant.

TABLE 140

FINANCE COSTS & ANNUAL REVENUE REQUIREMENTS

TREATMENT-TRANSPORT SYSTEM - CORE AREA

Loading Condition	Finance Cost	Total Annual Rev Req
Maximum	$246,000/yr	$1,027,000/yr
Medium	217,000	827,000
Minimum	147,000	517,000

From the above tables a comparison can be drawn between the standard piped interconnect system and the treatment-transport system. This comparison is illustrated in Table 141, assuming the Grimes site is adopted for terminal treatment.

Analysis of Table 141 indicates that the treatment-transport system will be more costly than a straight piped system, although the differential is not great for the maximum flow conditions. As the loading conditions lessen, the cost disadvantage to the treatment transport system increases, indicating the concept is primarily applicable to large loadings. Other advantages and disadvantages of each system are discussed in more detail in the next section, as is the comparison of the full interconnected Core Area system to the partially interconnected system.

TABLE 141

PIPED INTERCONNECT VERSUS TREATMENT-TRANSPORT

COMPARATIVE COSTS

Max Load	Total Cap	Local Cap	Oper	Finance	Total Ann Rev Req
Piped System	$21,820,000	$3,280,000	$750,000/yr	$238,000/yr	$ 988,000/yr
Treatment-Transport	22,655,000	3,400,000	780,000	247,000	1,027,000
Added Cost-Treatment-Transport	835,000	120,000	30,000	9,000	39,000
Med Load					
Piped System	17,770,000	2,670,000	560,000	194,000	754,000
Treatment-Transport	19,960,000	2,990,000	610,000	217,000	827,000
Added Cost-Treatment-Transport	2,190,000	320,000	50,000	23,000	73,000
Min Load					
Piped System	10,660,000	1,590,000	320,000	116,000	436,000
Treatment-Transport	14,765,000	2,210,000	370,000	147,000	517,000
Added Cost-Treatment-Transport	4,105,000	62,000	50,000	31,000	81,000

SECTION IX

COST AND ANCILLARY BENEFIT ANALYSES

The foregoing section presented detailed cost analyses of the many system alternatives available within the study area. This section summarizes the comparative costs and any ancillary benefits which may accrue from adoption of one alternative over another. For illustrative purposes the high loading option is used in each system comparison.

OUTLYING ADJACENT TOWNS

WASHBURN AREA - The Washburn analysis indicated that it would be somewhat more economical to construct a plant in Washburn rather than connecting to the Core Area at Presque Isle. This cost comparison is illustrated by the costs summarized in Table 142.

TABLE 142

COST COMPARISONS - LOCAL VERSUS INTERCONNECTED SYSTEM

Washburn Area

Cost Item	Local System Joint Plant	Interconnected System
Total Capital Cost	$ 2,314,000	$ 4,620,000
Local Capital Cost	524,000	947,000
Operating Cost	82,000/yr	98,000/yr
Total Annual Rev Req	$ 120,700/yr	$ 167,000/yr

The potential ancillary benefits to be gained by the interconnected system at Washburn are discussed in some detail in the cost analysis section. In the case of the Washburn interconnection, the benefits gained over the local system are not significant, and would be offset by local constraints imposed by such a system.

The Washburn analyses also indicated that a joint plant treating both the domestic and industrial wastewaters would be more economic than individual plants. This cost comparison is illustrated by the costs summarized in Table 143.

TABLE 143

COST COMPARISONS - JOINT VERSUS INDIVIDUAL PLANTS

Washburn Area

Cost Item	Joint System	Individual Systems
Total Capital Cost	$ 2,314,000	$ 2,700,000
Local Capital Cost	524,000	2,300,000
Operating Cost	82,800/yr	94,500/yr
Annual Revenue Requirements	120,700/yr	271,800/yr

The primary ancillary benefit gained by the joint plant system is administrative from the State or regional authority viewpoint, as only one plant must be monitored rather than two, and a single entity within the Washburn area would be responsible for the pollution control problem.

MAPLETON AREA - The Mapleton analyses indicated that it is more economic to construct a plant in Mapleton rather than interconnecting with the Core Area at Presque Isle. This cost comparison is illustrated by the costs summarized in Table 144.

TABLE 144

COST COMPARISONS - LOCAL VERSUS INTERCONNECTED SYSTEM

Mapleton Area

Cost Item	Local System	Interconnected System
Total Capital Cost	$320,000	$950,000
Local Capital Cost	48,000	142,500
Operating Cost	12,000/yr	12,000±/yr
Annual Revenue Requirements	15,500/yr	22,500±/yr

The primary ancillary benefit to be gained by interconnecting the Mapleton area with Presque Isle is the elimination of all direct treated wastewater discharges into the Presque Isle Stream above Presque Isle. This is of significance due to the water intake of the Presque Isle Water District. The interconnection would also eliminate one source of nutrients from the contemplated lower North Branch impoundment proposed by the Soil Conservation Service. Analysis of these benefits, however, indicate that they do not justify the added cost of transporting the small quantity of wastewater generated in Mapleton. If the waste volumes were larger, the benefits of interconnection could possibly outweigh the added costs.

FORT FAIRFIELD AREA - The Fort Fairfield analysis indicated that it is more economical to construct a plant in Fort Fairfield rather than interconnecting with the Core Area at Grimes Mill. This cost comparison is illustrated by the costs summarized in Table 145.

TABLE 145

COST COMPARISONS - LOCAL VERSUS INTERCONNECTED SYSTEM

Fort Fairfield Area

Cost Unit	Local System Joint Plant	Interconnected System
Total Capital Cost	$1,540,000	$2,825,000
Local Capital Cost	315,000	594,000
Operating Cost	63,000/yr	60,000±/yr
Total Annual Revenue Req	85,900/yr	103,200±/yr

The potential ancillary benefits to be gained by the interconnected system at Fort Fairfield are discussed in some detail in the cost analysis section. In the case of the Fort Fairfield interconnection, the benefits gained over the local system are not significant, and would be offset by local constraints imposed by such a system.

The Fort Fairfield analyses also indicated that a joint plant treating the domestic and industrial wastewaters would be more economical than individual plants. This cost comparison is illustrated by the costs summarized in Table 146.

TABLE 146

COST COMPARISONS - JOINT VERSUS INDIVIDUAL PLANTS

Fort Fairfield Area

Cost Item	Joint System	Individual System
Total Capital Cost	$1,540,000	$1,920,000
Local Capital Cost	315,000	1,495,000
Operating Cost	63,000/yr	80,600/yr
Annual Revenue Requirements	85,900/yr	189,100/yr

The primary ancillary benefits gained by the joint plant system is administrative from the State or regional authority viewpoint, as only one plant must be monitored and a single entity would be responsible for the area's pollution control program.

CORE AREA

The alternatives in the Core Area are more numerous and complex then in the outlying towns. The potential impact of non-direct ancillary benefits will also be greater in this area.

Analysis of the Core Area indicates that it will not be possible within reasonable economic limits to treat and dispose of the wastewater generated in the Easton area via discharge to the Prestile Stream, and that transfer to the Aroostook River Basin will be necessary. If this is accomplished, the analysis indicates it will be more economic to treat these wastes jointly with the combined Presque Isle municipal and Potato Service, Inc industrial wastes than to treat in Easton and discharge the effluent to the Aroostook River. This cost comparison is illustrated by the costs summarized in Table 147.

TABLE 147

COST COMPARISONS - JOINT VERSUS SEPARATE TREATMENT

Presque Isle-Easton Areas

Cost Item	Easton Alone Joint Plant	PQI-PSI Joint Plant	Total Ind Systems	Joint Easton-PSI-PQI
Total Cap Cost	$5,240,000	$4,920,000	$10,160,000	$10,055,000
Local Cap Cost	790,000	738,000	1,528,000	1,510,000
Oper Cost	232,000/yr	243,000/yr	475,000/yr	451,000/yr
Annual Rev Req	289,400/yr	297,500/yr	586,900/yr	560,000/yr

The primary ancillary benefit gained by a single treatment plant for the Easton-Presque Isle area is the concentration of facilities at a single location for control and monitoring. Such concentration will also facilitate attempts to process and utilize the excess biological solids.

Analysis of a full regional interconnected system for the Core Area indicates that it would be less costly to construct a single plant than to construct two plants, one at Presque Isle and one at Caribou. For illustrative analysis terminal facilities at Grimes Mill has been assumed. Of the two methods of interconnecting studied, the standard piped system would be more economic than the treatment-transport system. The comparative costs of the regional Core Area systems is illustrated in Table 148.

TABLE 148

COMPARATIVE COSTS - REGIONAL SYSTEMS - CORE AREA

Cost Item	Two Plant System	One Plant System Piped Interconnect	One Plant System Treatment-Transport
Total Capital Cost	$17,530,000	$21,820,000	$22,655,000
Local Capital Cost	2,635,000	3,280,000	3,400,000
Operating Cost	820,000	750,000/yr	780,000/yr
Annual Rev Req	1,001,000/yr	988,000/yr	1,027,000/yr

Analysis of Table 148 indicates that while the fully interconnected system for the Core Area has a higher cpaital cost than the two plant system, the total annual revenue requirements are essentially the same for either system. The treatment-transport system has a slightly higher annual revenue requirement than the standard piped system. Within the accuracy of preliminary design estimates, however, the total local annual revenue requirements for each system are essentially the same.

With the overall local cost requirements for each option of Core Area facilities about the same, careful analysis must be given to other non-direct cost factors which may affect system selection. The key element in any pollution control program is the enhancement of the receiving water, in this case the Aroostook River.

The companion River Basin Planning Report includes a detailed discussion of water quality and the impact of waste loads on the river. The reader is referred to that report for background material. The key aspects of this work are summarized herein as they apply to the Core Area facilities.

The Aroostook River is Class C from Washburn to the Canadian border, except for a 3 mile stretch above the Caribou municipal water supply intake. This classification is currently being violated on frequent occasions. Studies of the river indicate that it has an excellent assimilative capacity in all reaches except at the headpool of the dam in Caribou and at the Tinker dam headpool down river from Fort Fairfield. Except for these two locations, the river can readily accept the treated effluents with very little lowering of oxygen content. Within these pools, however, effluent will have a marked effect on the oxygen content. The "C" classification calls for a DO level of 5.0 mg/l for salmon and trout waters and 4.0 for non-salmon and trout waters under low river flow conditions (7 days low flow - 10 year return frequency).

The Aroostook River above Washburn is an excellent trout stream. It is expected that this category would certainly apply downstream from Washburn to the Tinker headpool in the vicinity of Fort Fairfield. The question of the Tinker headpool being classified as salmon and trout water may be debated. Prior to dam installations on the Saint John and Aroostook River, Atlantic salmon migrated into the Aroostook River. It is currently impossible for sea run salmon to reach the Aroostook River above Tinker dam. If the problems of power dam and pool passage of salmon in the Saint John River are solved, and fishways are successfully installed in Tinker dam, the salmon and trout category would apply. To a great degree this question will be subject to international negotiation between the United States and Canada.

The impact of the treated effluent on the Aroostook River, for the maximum load conditions in the Core Area, is summarized in Table 149.

TABLE 149

DISSOLVED OXYGEN CONDITIONS - AROOSTOOK RIVER

Maximum Load Conditions

Core Area System	Est Min DO Caribou Pool	Est Min DO Tinker Pool
2 Plant System	6.6 mg/l	5.6 mg/l
1 Plant System	8.4	4.7 mg/l

From Table 149 it can be seen that the two plant system will increase the DO problem at Caribou and will lessen the problem at Tinker pool. The one plant system will eliminate the DO problem at Caribou but will increase the problem at Tinker.

The B-1 classification at Caribou dictates a minimum DO concentration of 75% of saturation. At 20°C this value is 6.9 mg/l. It can be seen that with maximum load conditions, the two plant system may not meet the B-1 classification under design river flow. The single plant system is well above minimum at Caribou. The DO minimum at Tinker is well above either the 4.0 or 5.0 mg/l classification limit with the two plant system, if adopted. If the single plant system is adopted the DO level at Tinker may drop below the classification limit if a 5.0 mg/l minimum is adopted, but will be above classification if a 4.0 mg/l minimum is adopted.

The above discussions indicate that whichever system is adopted for the Core Area, the capacity of the river will be taxed to its limit at Caribou, or Tinker at maximum load conditions. It is doubtful, however, if actual discharges to the river will reflect maximum load conditions, as certain in-plant modifications have already been acconplished by industry. If a lesser, but moderately high, load is assumed, the river studies indicate that the classification requirements can be met. Recognizing that river studies to date are based on limited data, however, provisions should be made for maintaining the classifications if industrial expansion occurs. This can be accomplished by either increasing BOD removals through advanced waste treatment methods, by diverting a portion of the effluent into holding basins during drought conditions, instream aeration at critical conditions, or a combination of the three. The holding pond diversion system will be the most economical. These facilities could be installed at either the Potato Service, Inc site or the Grimes Mill site, but not at the in-town Caribou site.

The Caribou municipal water supply is taken from the Aroostook River above the Caribou dam. Although treated by sand filtration, the quality of the drinking water has been quite marginal from an esthetic standpoint. The water company has also encountered operational difficulties caused by slime and other organic growths. Improvement in raw water quality by installation of pollution control plants will certainly improve the Caribou water quality. The key question, however, is how much improvement would be achieved by carrying the Presque Isle-Easton wastewater below the intake than would occur if the treated effluent were placed in the river upstream? This question cannot be precisely answered in quantitative terms.

The operating problems at the Caribou water plant are for the most part caused by organic growths which inturn are probably related to nutrient conditions in the raw water. This is a very complex phenomenon and the critical nutrients for any particular organism is difficult to determine. Disposal of the treated Presque Isle-Easton effluent above the intake may or may not solve the problems at the Caribou water treatment plant. If the effluent were carried below the intake, the maximum protection possible would be provided.

It is difficult to evaluate the Caribou water supply problems in terms of firm costs. It is understood, however, that the Caribou Water Company has studied alternative supply sources, and none appear available at a reasonable cost. It is likely that capital improvements are going to be required at the Caribou Water Company plant in the relatively near future.

The extent of this capital investment will depend to some extnet on the raw water quality. If an entire new filtration plant is required, an investment of 2 to 3 million dollars could be anticipated. If a better raw water quality would allow rennovations to the existing facilities, this cost could possibly be reduced to about 1/2 million dollars. While these costs are approximate at best, they do indicate a potential significant cost benefit at Caribou, with the one plant system.

The Aroostook River is a transboundary river tributary to the Saint John River. As such its water quality will likely be subject to international negotiation between the United States and Canada. It is understood that intensive studies of the Saint John Basin are about to begin. The key water quality problem in the United States section of the river is oxygen content. However, one of the prime Canadian concerns is the nutrient conditions as they may affect the extensive power pools at Beechwood and Mactaquac. It is possible that future international negotiations may dictate that nutrient, primarily phosphorous, removal facilities be installed at plants along the Aroostook River. If this were necessary, it would prove more economical to achieve such removals at one large plant than at two separate plants. As space is available at Grimes, this factor would favor the one plant system. It should be noted that this factor would also tend to favor the standard piped system over the treatment-transport concept, as the latter inherently will have an effluent discharge at in-town Caribou.

Recreational uses of the river should also be considered. With installation of treatment facilities, the fisheries potential of the river below Presque Isle will increase. If the single plant system is adopted, there is no reason why the river between Washburn and Grimes Mill could not support an active trout fishery. This will have significant recreational value to the people of Presque Isle and Caribou. If the two plant system is adopted, it may also be possible to establish this fishery, although the water quality will not be as inducive to this program as the single plant system. It is not possible to equate this benefit in dollar terms.

Although the dissolved oxygen content of the river above Grimes Mill could support a B-1 or a B-2 classification, it is not likely that this will be possible because of the coliform bacteria counts. It is likely that the urban runoff from the Presque Isle and Caribou areas will contribute sufficient coliform bacteria to the water to preclude the higher classifications, at least until full separation of storm water is achieved in both cities. Thus, contact recreation, such as swimming, below Presque Isle will be limited. This situation is independent of which Core Area system is adopted.

Another factor which must be weighed in system selection in the Core Area is that of service area and economic growth of the communities involved. The interconnected system will provide a basic interceptor facility along the Aroostook River from Potato Service, Inc to Caribou, some 11 miles. As discussed in the companion Basin Planning Report, this area is located where development pressures will occur. It is served by US Route 1, the Bangor and Aroostook railroad, and the City of Caribou on the north and Presque Isle on the south. The provision of basic sewer service to this area will certainly enhance its development potential. Location of industrial development in a central area, capable of being served, is much more desirable than scattered development in unsewerable areas.
The provision of basic sewer service in this area will certainly enhance its development potential.

If the two plant system were adopted in the Core Area, the section between Presque Isle and Caribou could be served, provided additional interceptor sewers were built. If a service area equal to that obtained by the one plant system were to be achieved, an investment of 3 million dollars or more would be required. If a significant portion of the area between Presque Isle and Caribou, from the river to Route 1, develops and requires sewer services in the forseeable future, the added costs of interceptor sewers should be considered in economic evaluation of Core Area systems. If development is to be limited so sewer service will not be required, these costs would not be weighed in system selection.

The handling and disposal of excess biological solids must also be considered in Core Area system evaluation. As discussed earlier, initial programs will call for disposal, in conjunction with municipal solid waste disposal systems, in sanitary landfills. Effort should be made, however, to utilize the organic and nutrient value of this material as a soil conditioner. This will require certain storage and processing facilities if usage of this material is developed. Such facilities can more easily be concentrated at a single large plant than at two sites. If usage of this material cannot be developed and incineration is ultimately required, this too can be best accomplished at a single site.

The main potential esthetic problem with the piped interconnect system is that of possible odors at the Caribou end of the conduit. Special design considerations must be applied to overcome this problem. While a potential problem, it should not be insurmountable.

SECTION X

DISCUSSION

Extensive discussions have been included in the design, cost analysis, and other sections as appropriate. This format is helpful in a report of this magnitude as it allows discussion with adjacent data. Thus, this section will be limited to more general discussion.

The Northern Maine Regional Planning Commission must evaluate the data in this report and that contained in the companion Basin Planning Report. Upon such evaluation, a basin plan must be adopted in which both the selected physical system described herein, and the selected institutional system described in the Basin Planning Report is defined as the basis for project implementation. This is a large task and involves technical, economic, administrative, and political considerations, the mixture of which must be determined by the Commission. This report provides the basic technical and cost data for the Commissions evaluation. Data on the general economic, institutional, legal, and administrative considerations are presented in the Basin Planning Report. Based on these reports and the local knowledge of the Commission, implementation decisions must be made.

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SECTION XI

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Analysis, Treatment-Transport System - Barry A. Patrie, PE
In-plant Sampling and Analysis - James S. Atwell, PE
Hydraulics - Charles Horstmann, PE
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SECTION XII

LITERATURE REVIEW AND REFERENCES

POTATO PROCESSING WASTE CHARACTERISTICS

The processing of potatoes into frozen food products and potato starch is the primary industry of the Aroostook River-Prestile Stream watersheds of Northern Maine. Being a wet process type of industry, it creates the major pollution problems which must be solved if the water quality standards established by State and Federal authorities are to be met. To assist in the evaluation of waste loads to be encountered in the regional treatment system, a review of applicable literature was undertaken. This review is summarized in the following paragraphs, closing with a listing of references.

FLUMING AND WASHING - At most processing plants in the study area the potatoes are transferred from storage to processing by a water flume system. This system also serves to either fully or partially wash the potato. In this transport-wash system the carriage water picks up an appreciable pollution load. Water usage in the flume system varies greatly depending largely on the amount of recycling used. Wolters[1] reports European practice generates 800 to 1400 gallons per ton of potatoes in the flume and about 500 to 700 gallons per ton in a separate wash cycle. Adler[2] suggests values of 1000 to 1800 gallons per ton in Europe for fluming and washing. Jordan[3] reports 1450 to 1950 gallons per ton in a combined flume-wash system in Aroostook County, Maine. Wolters[1] indicates a BOD loading in the flume-wash water of 2 to 6 pounds per ton of potatoes. Jordan[3] reported up to 10 pounds BOD per ton of potato. Suspended solids in the flume water will vary greatly with the season and with in-plant removal practice.

PEELING - The peeling process has been the major source of organic waste load generated at a potato processing plant. Cooley[4] reports a wastewater production of 625 gallons per ton of potatoes in the steam peel process and 715 gallons per ton in the lye peel process. Corresponding BOD values were reported at 32.6 and 40.0 pounds per ton of potatoes respectively. Solids, COD and pH data were also reported. Sijbring[5] reported European values for steam peeling at 2.63 to 3.26 cubic meters per ton and BOD values of 79 to 114 population equivalents per ton. Corresponding values for lye peel operations was estimated at 3.82 to 5.48 cubic meters per ton and population equivalents of 240 to 345. Sproul, et al[6] reported BOD concentrations from lye peel operations at 1950 to 3550 mg/l. pH values of this flow were about 11.5

OVERALL PLANT WASTE FLOWS - Most published data deals with overall plant waste loads generated during processing. Sproul et al[6] report overall waste flows from a lye peel french fry plant at 2520 gallons per ton and 52 pounds BOD per ton. This data was taken prior to extensive in-plant conservation. After such conservation measures were initiated Sproul reports corresponding waste loads of 2310 gallons per ton and 22 pounds

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BOD per ton of potato. Flume loads were not included in data given. Olson[7] reported on waste loads from potato flake plants. A lye peel plant indicated a BOD of 1265 mg/l, suspended solids of 1294 mg/l and a pH of 10.8. A steam peel plant indicated a BOD of 701 mg/l, suspended solids of 297 mg/l and a pH of 7.1. Jordan[3] analyzed waste loads from a steam peel french fry plant and found flows ranging from 7500 to 9420 gallons per ton, BOD of 36 to 44 pounds per ton and suspended solids of 53 to 64 pounds per ton. These values include flume and wash flows. Kueneman[8] reported flows of 2670 gallons per ton from a steam peel french fry plant with BOD values of 1788 mg/l and suspended solids values of 1574 mg/l. Atkins and Sproul[9] suggest average values for a screened waste from lye peel french fry operations at 4210 gallons per ton, a BOD of 51 pounds per ton, and suspended solids values of 61 pounds per ton.

Dostal[10] reports on average discharges from three Idaho french fry processing plants as follows: Flow = 4200 gallons per ton, BOD = 90 pounds per ton, suspended solids = 110 pounds per ton, phosphorous = 0.6 pounds per ton, and nitrogen = 3.5 pounds per ton. Industry averages were suggested as 4200 gallons per ton, 50 pounds BOD per ton, and suspended solids 60 pounds per ton. Cooley[4] reported overall waste flows from a potato flake plant as follows: Flow = 1540 gallons per ton, BOD = 38.4 pounds per ton, total solids = 125.6 pounds per ton, suspended solids = 16.4 pounds per ton, and a pH of 5.2.

Ambrose and Reiser[11] indicated a waste water flow of 2755 gallons per ton of potato from potato starch plants carrying about 35 pounds BOD per ton, excluding pulp. Hinkley[12] estimated a BOD generation of about 57 pounds per ton of potatoes at a Maine starch plant. Caron[13] estimates flows from a potato starch plant at 2160 to 3600 gallons per ton of potatoes.

TREATMENT OF POTATO PROCESSING WASTES

In recent years considerable effort has been directed at treatment of potato processing wastewaters. Primary treatment for solids removal has been proven successfully in the field at a number of plants. Biologic treatment of potato processing wastewaters has received considerable study, but actual full scale plant data is limited. The literature review included data assembly on both the primary and biologic phases of treatment.

SCREENING - Ballance[14] reported on efficiencies of rotary screens in treating potato processing wastes. The most common screen sizes used were 20 and 40 mesh, with 120 mesh being the smallest practical. Suspended solids removal of 35%± were reported on a 20 mesh screen. Solids removals on a 20 mesh screen were found to be about 25% by Grames and Kueneman[15]. They also indicated that BOD reduction of 10 to 14% could be anticipated. Talburt and Smith[16] suggested suspended solids removals of 34% and BOD removals of 23% by screening at a french fry plant.

PRIMARY TREATMENT - Talburt and Smith[16] found that overflow rates of 800 to 1000 gal/sq ft/day would give 50 to 75% COD removal after screening. A tank depth of 10 to 12 feet was suggested. Filbert[17] found similar values. Kueneman[18] found that 50% BOD removal could be achieved by primary sedimentation at an overflow rate of 800 gal/sq ft/day. Detention times were 2 to 4 hours. The solids underflow from a lye peel plant was found to be 3.5% which could be conditioned to 5 to 8%. Solids underflow at a steam peel plant was found to be 5 to 6%. Kueneman[8], in other studies, reported a 93% suspended solid and a 70% BOD removal by primary clarification of a steam peel effluent. Surface loading was suggested at 600 to 800 gal/sq ft/day. Lye peel plant efficiencies were slightly less although a 66% COD removal was reported. At an overflow rate of 600 to 800 gal/sq ft/day, others reported the following removal efficiencies:

Pailthrop and Filbert[19] - 58% suspended solids, 55% BOD removal
Olson[7] - Potato Chip Plant - 92% suspended solids, 63% BOD removal
Potato Flour Plant - 83% suspended solids, 51% BOD removal
Dostal[10] - Lye Peel French Fry - 37% BOD removal

Jordan[3] reported on sedimentation column tests on processing wastes from a steam peel french fry plant. Excellent solids and BOD removals were indicated. Removal vs loading curves were presented in the Jordan Report. The Jordan[3] study also demonstrated that sedimentation of potato starch wastes was not effective with removals of only 21% suspended solids and 11% BOD.

PRIMARY SOLIDS DEWATERING - Dewatering of primary potato solids has been successfully accomplished by vacuum filtration. Grames and Kueneman[15] indicate that successful filtration is very pH dependent. With the pH between 6 and 8$\underline{+}$, the solids can be dewatered without extensive chemical conditioning. With such pH adjustment, usually by proper timing of solids withdrawal, a filter sludge of 12 to 16% solids can be obtained.

The solids content of the filtrate is suggested as 1000 to 1200 mg/l. A design filter loading of 5 pounds dry solids per hour per square foot of filter surface was suggested, although rates up to 15 pounds per hour per square foot have been realized. Jordan[3] reported on filter leaf tests on primary potato solids from a steam peel french fry plant. Again pH is shown to be critical. Sludge with a pH above 6.0 could be readily dewatered without chemical conditioning. If the pH drops below 6.0, conditioning with feric chloride was required for successful filtration. It was shown that the fresh solids pH of 6.9 would drop rapidly and would reach 4.9 within a few hours. Filter design loadings of 6.0 to 7.0 pounds per hour per square foot for a steam peel plant were suggested. The filtrate would contain 2000 to 3000 mg/l solids if conditioners were not used, and would fall to about 1000 mg/l if the solids were conditioned with 2% ferric chloride prior to dewatering. Solids contents of the filter cake were reported at 9.5 to 13.5% without conditioning, and 18 to 19% with conditioning. Additional data on form times were also given in the Jordan Report.

BIOLOGICAL TREATMENT - Extensive studies have been conducted on biological treatment of potato processing waters by the activated sludge system and variations thereof. The earliest significant work was conducted by Atkins and Sproul[9] in 1964. These studies indicated that BOD removals of over 90% could be achieved in a completely mixed activated sludge system treating potato wastes from a lye peel plant. Aeration times of 6 to 12 hours were studied with 8 hours suggested for design. It was demonstrated that pH adjustment was not necessary and the system could be acclimated to wastes from a lyle peel plant. Organic loadings of 200 to 400 pounds BOD per 1000 CF aeration capacity per day were found satisfactory. Mixed liquor suspended solids levels of 3000 to 4000 mg/l were used in the bench scale units. The Atkins and Sproul work also provided limited data on treatment by the contact stabilization method. A contact time of 1 hour and a reaeration time of 6 to 8 hours was suggested. BOD removals of about 80% were achieved.

Cornell, Howland, Hayes and Merryfield[20] reported on contact stabilization studies on potato wastes. A contact time of 1 hour and a reaeration time of 10 hours was suggested. Organic loads were up to 480 pounds BOD per 1000 cubic feet. BOD removals of 60% were achieved. This study suggested that nutrient addition may be necessary, and foaming problems could exist in such a system. Kintzel[21] reported good BOD removals on a potato starch wastewater treated by contact stabilization.

Jordan[3] reported on bench scale studies of completely mixed activated sludge systems applied to wastewaters from a steam peel french fry plant, and on this waste combined with a potato starch plant waste. Design recommendations from this work were as follows:

Aeration Time = 7 Hours
MLSS = 3000 mg/l
$K_s = 0.000425$
BOD Removal = 90%+ at 20°C
Temperature Relation: $(K_s)_T = (K_s)_{20} \times 1.05^{T-20}$
Solids Production: LB VSS/day = .65(LB BOD Removed)-.05(MLVSS)
Maximum Oxygen Uptake = 38 mg/l per hour

Jordan also reported on limited tests with aerated pond systems for potato processing wastes. Removals of up to 70% could reasonably be achieved by such a system.

Dostal[22] conducted studies on both anerobic and aerobic ponds following primary sedimentation of a lye peel processing plant effluent. Removals achieved in the primary clarifier confirmed present data. Test data on the covered anerobic phase of the system showed poor BOD removals. The aerobic phases of the system indicated fairly good BOD removals, although it was evident that final clarification would be required to achieve consistent results.

The R. T. French Co.[23] conducted extensive tests on a full scale activated sludge plant at Shelley, Idaho. The following summarizes the key design criteria and other data generated by the French Report:

Design Flow = 1.25 MGD
BOD = 14,100 LB/Day
Suspended Solids = 6620 LB/Day
Aeration Volume = 3.75 MG
Basin Depth = 16 Ft
Design Organic Load = 28 LB BOD/1000 Cubic Ft/Day
MLSS = 4000 Mg/l
Operating DO = 1.5 Mg/l
Overflow Rate - Secondary Clarifier = 325 Gal/Sq Ft/Day

Although significant operational difficulties were encountered in the French Plant, it was demonstrated that under normal conditions BOD removals of over 90% could be achieved. Effluent BOD was reduced to 85 mg/l unfiltered, and 40 mg/l filtered. An attempt was made to combine the clarification of the flume water with thickening of excess biologic solids.

This system did not function well and separate facilities are suggested. Buzzell et al[24] reported on a bench scale completely mixed activated sludge system treating potato starch wastes. It was concluded that nutrient addition was not required and 90% BOD removal could be achieved with organic loadings up to 420 lbs BOD/1000 cu ft/day. Foaming of the system was considered a problem. Michaelson[25] presented data on a full scale activated sludge plant which served to lessen organic loads prior to entry into a municipal system. Loadings were 400 lbs BOD/1000 cu ft/day with a detention time of 5 hours. The effluent BOD varied from 200 to 400 mg/l. A sludge thickening unit was used with a loading of 8 lbs solids/sq ft/day.

PRIMARY SOLIDS DISPOSAL AND UTILIZATION - Disposal of solids removed from primary clarifiers treating potato wastes may or may not present a problem, depending on the possible utilization of the material. Hinden and Dunstan[26] reported on the composition of waste potato solids as follows:

Total Organic Nitrogen as N = 1.002%
Carbon as C = 42.2%
Total Phosphorous as P = 0.083%
Total Sulfur as S = 0.082%
Volatile Solids = 95.2%

Guttormsen and Carlson[27] summarized the use of potato solids as cattle feed. Screenings and dewatered primary solids have been successfully used for this purpose. Lye peel solids must be fermented to reduce the pH to 7.0 or below prior to use. Potato solids are usually mixed with chopped hay or other feed in varying amounts. Potato solids have been sold for $3 per ton for feed purposes. Dickey et al[29] discussed use of potato pulp for feed to a variety of animals. The general composition of potato pulp was reported as follows:

		Dried	Wet
Moisture	=	11.83%	83.3%
Protein	=	5.71%	1.20%
Fat	=	0.40%	0.06%
Fibre	=	13.16%	1.06%
Nitrogen Free Extract	=	66.58%	13.80%
Minerals		2.58%	0.51%

DRY CAUSTIC PEELING - In an attempt to minimize organic wastewater loadings from the peeling operation, extensive work has been directed at development of a dry, or semi-dry caustic peeling process. Willard[29] reported on tests accomplished on such peeling systems. By utilizing a caustic dip system followed by infra red heat, it is possible to remove the peelings with a dry or semi-dry brush system. The system was reported to produce a pulp of 12 to 14% solids which could be pumped. Peel loss and caustic usage was compared favorably with conventional systems. Material could be used for feed if allowed to ferment to reduce pH.

Cornell, Howland, Hayes and Merryfield[30] prepared comparative data on the dry caustic peel system for Magnuson Engineers, Inc. makers of peeling equipment. The following comparative loads to the primary clarifier, excluding flume or washing operations, were suggested for a 40,000 pound per hour plant.

Dry Caustic.	11,700 LBS BOD/Day
Steam Peel	18,400 LBS BOD/Day
Lye Peel 13% Loss.	29.500 LBS BOD/Day
Lye Peel 18% Loss.	36,300 LBS BOD/Day

This report also presented approximate comparative cost data for pollution control systems.

SUGAR BEET PROCESSING

A secondary industry which is in the process of development in northern Maine is refining sugar from beets. Although currently not operative, the impact of this industry on any regional pollution control program must be considered. The Federal Water Pollution Control Administration Report[31] on the beet sugar industry summarized the existing condition in the industry and the current methods being used or explored for pollution reduction. This report indicates that BOD production in United States plants ranged from 10 to 33 pounds BOD per ton of beets processed, with an average of 15 pounds per ton. These values exclude the organic loads from mud lime and silo drainage. It was also reported that beet plant effluents contain high bacteria counts, including salmonella, and this may constitute a significant health problem. The report suggested that large pollution reductions could be achieved by in-plant water conservation and recycling. While such procedures could create some operating problems and would require treatment in the recycle system, it was shown that BOD generation could be reduced to 1 pound per ton of beets, or possibly less if an essentially complete reuse system were adopted. Biological treatment of beet wastes has not been used extensively, due in part to the short processing season of 80 to 160 days per year. Pilot plant studies of activated sludge systems show good results, although nutrient additions may be required.

Lof and Kneese[32] undertook an extensive economic analysis of water conservation measures in the beet processing industry. They indicated a potential BOD generation of nearly 40 pounds per ton of beets, including lime cake slurry and silo drainage. Without the latter elements, now common in United States practice, a value of 16.8 pounds BOD per ton of beets is given. Lof and Kneese further suggest that this value could readily be reduced to about 6 pounds per ton by in-plant conservation and limited treatment, and could ultimately be reduced to 1.2 pounds per ton with extensive recycling of treated waters. Camp Dresser and McKee[33] in a report to the Town of Easton, Maine, based load estimates on a BOD generation of 4 pounds per ton of beets based on a plant capacity of 4000 tons per day. Average flow was estimated at 2000 gpm with maximum flows approaching 3000 gpm during the processing season.

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SECTION XIII

COMPUTER DESIGN PROGRAMS

A set of waste treatment plant design programs were written specifically for this study for evaluating the various treatment process configurations. The programs were written in FORTRAN IV programming language for the IBM 1130 Computing System with 16,000 words of core, but could easily be modified to run on other systems. Input consists of the basic design criteria for process and structural design. Output is composed of design and quantity data for each design configuration. System diagrams for the treatment processes appear in Figure 65.

The presentation of this documentation assumes that the reader is familiar with waste treatment facility design and Fortran programming.

DESCRIPTION OF PROGRAMMING - The program set is made up of two mainline programs (PRIME and SECON) and one subroutine (QUATY) called by both mainlines.

PRIME and SECON compute the process design data for primary (sedimentation) and secondary (activated sludge) treatment processes respectively; QUATY computes earthwork and concrete for both.

No permanent storage or file space is required. The programs run under Version 2, Modification 10 of the IBM Monitor System.[1]

DESCRIPTION OF INPUT - Numeric data must contain a decimal point or be right justified.

PRIME

Description	S.C. Unit	Inclusive Card Columns	Format
CARD NO. 1 - STANDARD DATA			
Suspended Solids in Sludge Outflow	(%)	1-7	F7.0
Side Water Depth	(ft)	8-14	F7.0
Bottom Slope	(in/ft)	15-21	F7.0
Free Board	(ft)	22-28	F7.0
Sidewall Thickness	(in)	29-35	F7.0
Bottom Slab Thickness	(in)	36-42	F7.0
Filter Loading	(lbs/hr)	43-49	F7.0
Suspended Solids in Solids Input	(%)	50-56	F7.0
Gravel Base Mat	(ft)	5. -63	F7.0
Suspended Solids Removal	(%)	64-70	F7.0
BOD Removal	(%)	71-77	F7.0

1. IBM 1130 Disk Monitor System, Version 2, Programmer's and Operator's Guide, File No. 1130-36, Order No. GC26-3717-8

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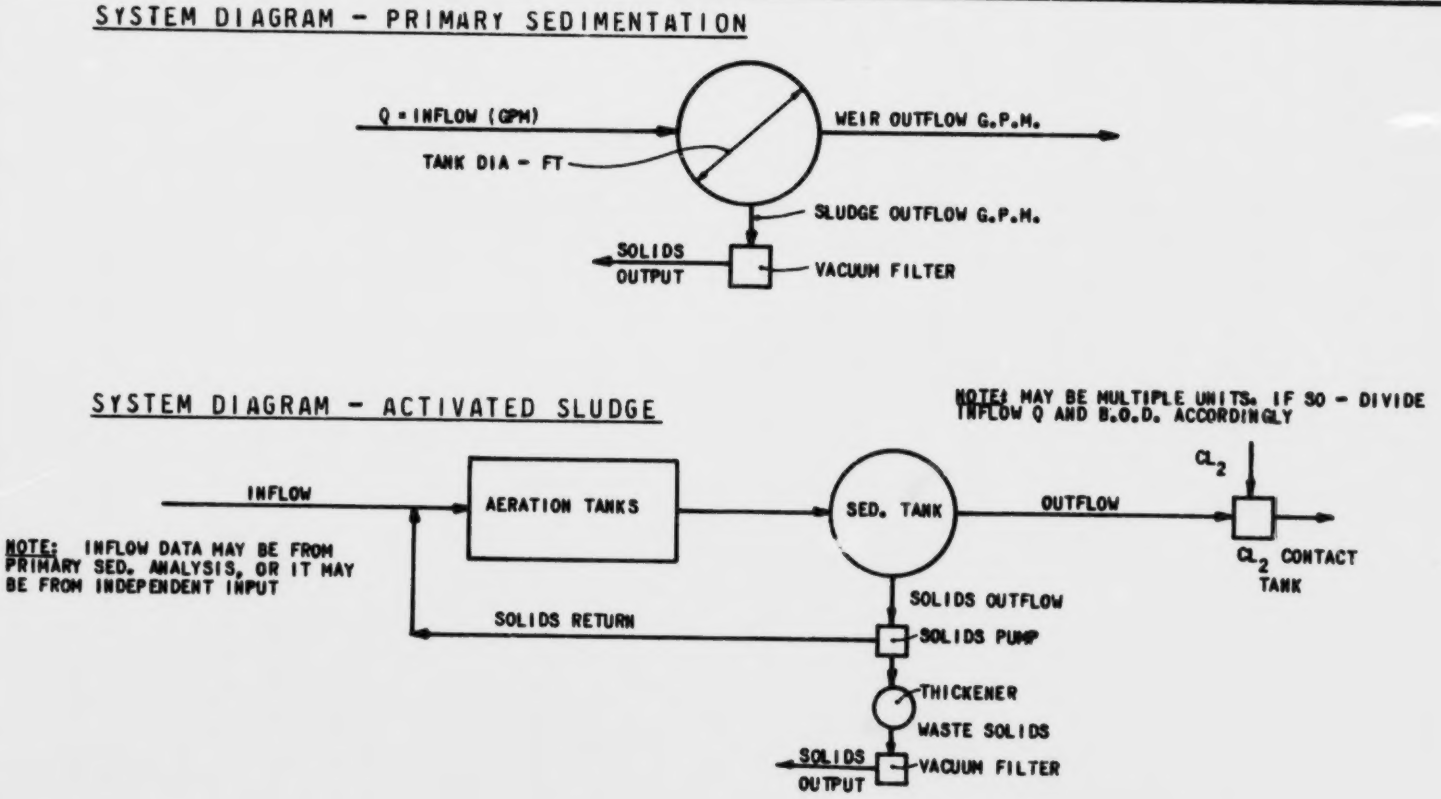

SYSTEM DIAGRAM - PRIMARY SEDIMENTATION

Q = INFLOW (GPM)
TANK DIA - FT
WEIR OUTFLOW G.P.M.
SLUDGE OUTFLOW G.P.M.
SOLIDS OUTPUT
VACUUM FILTER

SYSTEM DIAGRAM - ACTIVATED SLUDGE

INFLOW
AERATION TANKS
SED. TANK
OUTFLOW
CL_2
CL_2 CONTACT TANK
SOLIDS OUTFLOW
SOLIDS RETURN
SOLIDS PUMP
THICKENER
WASTE SOLIDS
SOLIDS OUTPUT
VACUUM FILTER

NOTE: INFLOW DATA MAY BE FROM PRIMARY SED. ANALYSIS, OR IT MAY BE FROM INDEPENDENT INPUT

NOTE: MAY BE MULTIPLE UNITS. IF SO - DIVIDE INFLOW Q AND B.O.D. ACCORDINGLY

FIGURE 65
TREATMENT SYSTEM DIAGRAM

Description	S.C. Unit	Inclusive Card Columns	Format
CARD NO. 2 - VARIABLE DATA			
Inflow	(gpm)	1-7	F7.0
Suspended Solids Inflow	(mg/l)	8-14	F7.0
BOD Inflow	(mg/l)	15-21	F7.0
Surface Loading	(gal/sf/day)	22-28	F7.0
Filter Operation	(hrs/day)	29-35	F7.0
Height of Earth Above Base Slab	(ft)	36-42	F7.0
Run Description		43-80	A1

NOTES:

1. Data Order - One card no. 1 (standard data) followed by any number of card no. 2's (variable data) with a blank at the end.

2. Percentages are expressed as decimal values, e.g. 52% = 0.52

SECON

Description	S.C. Unit	Inclusive Card Columns	Format
CARD NO. 1 - STANDARD DATA			
Oxygen Uptake	(mg/L/hr)	1-7	F7.0
Volatile Sus. Sol. in Mixed Liquor Suspended Solids	(%)	8-14	F7.0
Solids Growth Constants "a"		15-21	F7.0
Suspended Solids in Solids Outfall	(%)	22-28	F7.0
Surface Loading to Thickener	(gal/sf/day)	29-35	F7.0
Filter Loading Rate	(lbs/hr/sf)	36-42	F7.0
Solids Content of Solids Output	(%)	43-49	F7.0
Side Water Depth, Sedimentation Tank	(ft)	50-56	F7.0
Bottom Slope Sedimentation Tank	(in/ft)	57-63	F7.0
Side Slope Inside Aeration Tank		64-70	F7.0
Side Slope Outside Aeration Tank		71-77	F7.0
Card No. (1)		80	F7.0
CARD NO. 2 - STANDARD DATA			
Freeboard, Aeration Tank	(ft)	1-7	F7.0
Freeboard, Sedimentation Tank	(ft)	8-14	F7.0
Solids Growth Constant "b"		15-21	F7.0
Sedimentation Tank Sidewall Thickness	(in)	22-28	F7.0

Description	S.C. Unit	Inclusive Card Columns	Format
Card No. 2 - Standard Data (Cont)			
Sedimentation Tank Base Slab Thickness	(in)	29-35	F7.0
Sedimentaton Tank Gravel Thickness	(ft)	36-42	F7.0
Aeration Tank, Gravel Mat Thickness	(ft)	43-49	F7.0
Aeration Tank, Concrete Lining Thickness	(in)	50-56	F7.0
Rate of Biological Reaction		57-63	F7.0
Minimum Expected Temperature	(°c)	64-70	F7.0
Width of Top Berm	(ft)	71-77	F7.0
Card No. (2)		80	
CARD NO. 3 - STANDARD DATA			
CL_2 Contact Time	(min)	1-7	F7.0
CL_2 Contact Tank Depth	(ft)	8-14	F7.0
Height of Earth Above CL_2	(ft)	15-21	F7.0
CL_2 Contact Tank Base Slab Thickness	(in)	22-28	F7.0
CL_2 Contact Tank Wall Thickness	(in)	29-35	F7.0
CL_2 Contact Tank Free-Board	(ft)	36-42	F7.0
Hours Pumping Solids	(hrs)	43-49	F7.0
Filter Hours Operation	(hrs)	50-56	F7.0
Card No. (3)		80	
CARD NO. 1 - VARIABLE DATA			
Inflow	(gpm)	1-7	F7.0
Inflow BOD	(mg/L)	8-14	F7.0
Rate of Solids Return	(%)	15-21	F7.0
Aeration Detention	(hrs)	22-28	F7.0
Organic Loadings	(lbs of BOD)	29-35	F7.0
Mixed Liquor Suspended Solids	(mg/L)	36-42	F7.0
Hydraulic Loading, Secondary Clarifier	(gal/sf/day)	43-49	F7.0
Water Depth Aeration Tank	(ft)	50-56	F7.0
Length/Width Ratio		57-63	F7.0
Design Temperature	(°C)	64-70	F7.0
Earth Height Above Concrete Base	(ft)	71-77	F7.0
Card No. (1)		80	
CARD NO. 2 - VARIABLE DATA			
CL_2 Dosage	(mg/L)	1-7	F7.0
Earth Height Above Aeration Tank Base	(ft)	8-14	F7.0
Description		15-52	A1
Card No. (2)		80	

NOTES:

1. Data Order - One set of standard data (cards 1, 2, and 3) followed by as many sets of variable data (cards 1 and 2) as desired followed by a blank.

2. Percentages are expressed as decimal values, e.g. 50% = 0.52

OPERATING INSTRUCTIONS - Under the IBM Monitor System these programs may be stored and called by execution cards or executed under the load and go concept. The following execution cards presume that the program will be stored.

Execution Cards

PRIME

```
// XEQ PRIME

     Card No. 1 (standard data) one per run
     Card No. 2 ( variable data) as many as desired
     Blank
```

SECON

Standard Monitor input/output error messages will alert the user to errors in data fields. No effort has been made to edit the data within the program logic.

VARIABLE LIST

DESCRIPTION	VARIABLE NAME
Inflow in GPM	Q
Suspended solids inflow in MG/L	SSIN
BOD inflow in MG/L	BODIN
Surface loading in GAL/SF/DAY	SURFL
Suspended solids removal in %	SSREM
BOD removal in %	BODRE
Suspended solids in sludge outflow in % solids	SSSLU
Side water depth in feet	SWDEP
Bottom slope in inches/foot	BOTSL
Freeboard in feet	FREEB
Sidewall thickness in inches	SWTHK
Bottom slab thickness in inches	BOTST
Filter loading in LBS solids/hr	FILLD
Suspended solids in solids output in % solids	SSOUT
Period of filter operation in hours/day	PERID
Height of earth above top of base slab in feet	HTEAR
Gravel base mat thickness in feet	GRAVB
Surface area	SA

Tank Diameter	TKDIA
Tank Volume	TKVOL
Detention Time	TIME
Weir Loading	WLOADS
Suspended solids in weir outflow	SOLWR
Solids to sludge	SOLSG
Sludge outflow	SLOUT
Filter solids output	FILSO
Filter area	FILAR
BOD in weir outflow	BODWO
Earth excavation	EE
Gravel mat	GM
Base slab concrete	BC
Wall concrete	WC
Backfill	BF
Disposable excavation	DE
Peripheral underdrains	PUD

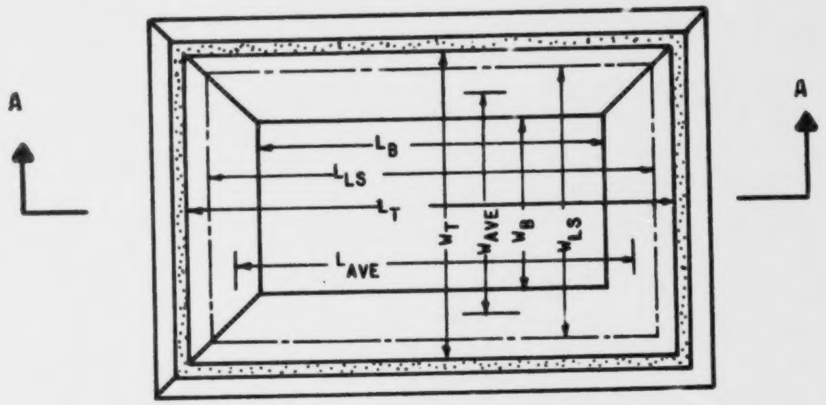
SECON

DESCRIPTION	VARIABLE NAME
Oxygen uptake in LSB O_2/LB BOD applied	A (1)
Percent volatile suspended solids in mixed liquor suspended solids in percent	A (2)
Solids growth constant "a"	A (3)
Suspended solids in solids outflow in percent	A (4)
Surface loading to thickener in gallons of sludge/SF/DAY	A (5)
Loading rate to filter in LBS/Hour/SF	A (6)
Solids content of solids output in percent	A (7)
Side water depth in sedimentation tank in feet	A (8)
Bottom slope of sedimentation tank in inches/foot	A (9)
Side slope, aeration tank, inside	A (10)
Side slope, aeration tank, outside	A (11)
Freeboard - aeration tank in feet	A (12)
Freeboard - sedimentation tank in feet	A (13)
Solids growth constant "b"	A (14)
Thickness of sedimentation tank sidewalls in inches	A (15)
Thickness of sedimentation tank base slab in inches	A (16)
Thickness of sedimentation tank gravel mat in feet	A (17)
Thickness of aeration tank gravel mat in feet	A (18)
Thickness of aeration tank concrete lining in inches	A (19)
Rate of biologic reaction	A (20)
Minimum expected temperature °Centigrade	A (21)
Width of top berm in feet	A (22)
CL_2 contact time in minutes	A (23)
Depth of CL_2 contact tank in feet	A (24)
Height of earth above base slabs of contact tank in feet	A (25)
Thickness of CL_2 contact tank base slabs in inches	A (26)
Thickness of CL_2 contact tank walls in inches	A (27)

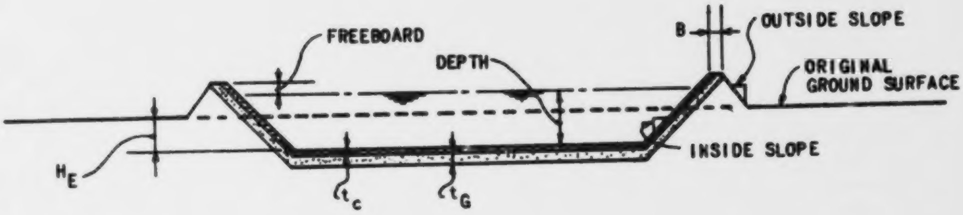
Freeboard - CL_2 contact tank	A (28)
Hours pumping solids	A (29)
Filter hours operation	A (30)
Inflow in gallons per minute	B (1)
BOD in inflow - MG/L	B (2)
Rate of solids return in percent of inflow	B (3)
Aeration detention time in hours	B (4)
Organic loading in LBS BOD/1000 CF aeration volume	B (5)
Mixed liquor suspended solids in MG/L	B (6)
Hydraulic loading of secondary clarifier in GAL/SF/DAY	B (7)
Water depth in aeration tank	B (8)
Ratio of lenfth to width in aeration tank	B (9)
Design temperature °Centigrade	B (10)
Height of earth above concrete base in feet	B (11)
CL_2 dosage in MG/L	B (12)
Height of earth above aeration tank base	B (13)
Aeration tank volume in cubic feet	C (1)
Organic load in LBS BOD/1000 CF/DAY	C (2)
Required aeration volume in cubic feet	C (3)
BOD to mixed liquor suspended solids ratio	C (4)
Oxygen required in LBS/Hour	C (5)
Solids return rate in GPM of solids returned	C (6)
Percent BOD removal in percent at 20° C	C (7)
BOD percent removed at design temperature in percent	C (8)
Biological solids generation in LBS/DAY	C (9)
Excess solids volume in gallons/day	C (10)
Excess solids pumping rate in GPM	C (11)
Thickener surface area in square feet	C (12)
Filter area in square feet	C (13)
Solids output in cubic yards/day	C (14)
Sedimentation tank surface area in square feet	C (15)
Sedimentation tank diameter	C (16)
Sedimentation tank volume	C (17)
Detention time in hours	C (18)
Weir loading in gallons/LF/day	C (19)
BOD in outflow in MG/L	C (20)
Aeration horsepower	C (21)
Earth excavation	C (22)
Gravel mat	C (23)
Base slabs concrete	C (24)
Wall concrete	C (25)
Backfill	C (26)
Disposable excavation	C (27)
Peripheral under drains	C (28)
Averate area at one-half side water depth	C (29)

Note: See Figure 66 for following dimensions:

W_{ave} in feet	C (30)
L_{ave} in feet	C (31)
L_b in feet	C (32)
W_b in feet	C (33)
L_{Ls} in feet	C (34)
W_{Ls} in feet	C (35)
L_t in feet	C (36)
W_t in feet	C (37)
Volume of excavation in cubic yards	C (38)
Volume of embankment in cubic yards	C (39)
Volume of spoil in cubic yards	C (40)
Volume of concrete in cubic yards	C (41)
Volume of gravel mat in cubic yards	C (42)
Volume of CL_2 contact tank in cubic feet	C (43)
Area of CL_2 contact tank in square feet Note: Length to width ratio assumed L=2W	C (44)
Volume of CL_2 contact tank excavation in cubic yards Note: 1:1 excavation slope is assumed	C (45)
Volume of contact tank base concrete in cubic yards	C (46)
Volume of contact tank wall concrete in cubic yards	C (47)
Volume of contact tank gravel mat in cubic yards	C (48)
Volume of contact tank backfill in cubic yards	C (49)
Volume of contact tank spoil in cubic yards	C (50)
Volume of chlorine required in LBS/DAY	C (51)


A
A
L_B
L_LS
L_T
L_AVE
W_T
W_AVE
W_B
W_LS

PLAN


FREEBOARD
DEPTH
B
OUTSIDE SLOPE
ORIGINAL GROUND SURFACE
INSIDE SLOPE
H_E
t_c
t_G

SECTION A - A

FIGURE 66
AERATION UNIT CONFIGURATION
QUANTITY ESTIMATE

```
// JOB
// FOR
*EXTENDED PRECISION
*ONE WORD INTEGERS
*LIST ALL
*IOCS(CARD,DISK,1403 PRINTER)
      DIMENSION  A(30),B(13),DESC(38),C(51)
C     INPUT SECTION
C     READ STANDARD INFORMATION
      READ(2,2) A
    2 FORMAT(11F7.0)
C     READ VARIABLE DATA
    3 READ(2,4)(B(J),J = 1,11)
    4 FORMAT(11F7.0)
      READ(2,6) B(12),B(13),DESC
    6 FORMAT(2F7.0,38A1)
      IF (B(1))200,200,7
C     AER  TANK  VOL
    7 VOL  = ( (B(1) + B(3)* B(1))/7.48) * 60. * B(4)
C     ORGANIC LOAD
      C(2) = ( B(2) *  B(1) * 8.33 *1440. )/(1000000. *  VOL /1000.)
C     RECUIRED AER. VOL
      C(3) = (B(1) * 1440. * B(2) * 8.33 * 1000.)/(1000000.* B(5))
      IF(C(2) - B(5)) 8,8,10
    8 C(1) = VOL
      GO TO 12
   10 C(1) = C(3)
C     BOD TO MLSS RATIO
   12 C(4) =  (B(1)* 1440 * B(2) * 8.33) /(1000000. * B(6) * 8.33 *(
     1C(1) * 7.48 / 1000000.))
C     OXYGEN REQ
      C(5) = A(1) * (B(2) * B(1) * 1440 * 8.33 /(1000000. * 24.))
C      SCLIDS RETURN RATE
      C(6) = B(1) * B(3)
C     PERCENT BOD REMOVAL
      C(7) = (100. * A(20) * B(6) * A(2) * B(4)) /( 1 +(A(20) * B(6) * A
     1(2) * B(4)))
C     PERCENT BOD REMOVAL AT DESIGN TEMP
      EXP = A(21) - 20
      ASD =  A(20) * 1.05 **     EXP
      C(8) = (100. * ASD * B(6) * A(2) * B(4))/(1+ ASD * B(6) * A(2)
     1 * B(4) )
C     BIOLOGIC  SOLIDS  GEN
      C(9) = ((A(3)*B(1)*1440.*B(2)*8.33*C(7)/100.)/1000000.)-(A(14)*B(6
     1)*8.33*C(1)*7.48*A(2)/1000000.)
C     EXCESS SOLIDS VOL
      C(10) = C(9)          * 7.48 /(62.4 * A(4))
C     EXCESS SOLIDS PUMP RATE
      C(11) = C(10) /(1440 * A(29)/24)
C     THICKENER SURF A
      C(12) =    C(11) * 1440 / A(5)
C     FILTER AREA
      C(13) = C(9) /(A(6) * A(30))
C     SCLIDS OUTPUT
      C(14) =  C(9) * 100. /(62.4 * A(7) * 27 *100.)
C     SECIMENTATION TANK SURFACE AREA
      C(15)  =  (B(1)  +  (B(3) * B(1))) * 1440./ B(7)
C     SED TANK  DIA
      C(16) = IFIX(((4*C(15)/3.1418)**.5)  +.5)
C     SED. TANK  VOL
      C(17)  =  C(15)*  ((A(8) + (A(8) +(C(16)* A(9)/24)))/2)
      WRITE(5,20)
   20 FCRMAT(1H0,51X,'VARIABLE  DATA (INPUT)')
      WRITE(5,22)
   22 FCRMAT(1H0,30X,'  INFLOW     S S      B O D    SURFACE    FILTER
     1   EARTH'  /1H ,40X,'  INFLOW    INFLCW   LOADING     OPS       HEI
     2GHT  '/1H ,30X,'  (GPM)      (MG/L)    (MG/L) (GAL/SF/DAY) (HRS/CAY
     3)  (FT)')
      WRITE(5,24) Q,SSIN,BODIN,SURFL,PERID,HTEAR
   24 FORMAT(1H0,27X,6F10.1////)
      WRITE(5,26)
   26 FCRMAT(1H0,57X,'OUTPUT'/1H0,'  SURFACE   TANK       TANK      DETEN
     1T    WEIR       S S        SOLIDS    SLUDGE   FILTER     FILTER    B
     2 O D ')
      WRITE(5,28)
   28 FCRMAT(1H , '   AREA    DIAM      VOL          TIME    LCADING    WE
     1IR        TO      OUTFLOW   SOLIDS     AREA     WEIR    '/1H ,52X,'U
     2UTFLOW   SLUDGE                 OUTPUT                 OUTFLOW')
      WRITE(5,30)
   30 FCRMAT(1H ,'  (SQFT)     (FT)    (CF)        (HRS)  (GAL/LF/DAY)(MG
     1/L)  (LBS/DAY)    (GPM)     (CY/DAY)    (SF)       (MG/L) ')
      WRITE(5,32)SA,TKDIA,TKVOL,TIME,WLOAD,SCLWR,SOLSG,SLOUT,FILSC,FILAR
     1,BCDWO
   32 FCRMAT(1H0,F7.1,10F10.1 ////)
      WRITE(5,34)
   34 FCRMAT(1H0,10X,'  EARTH          GRAVEL      BASE       WALL       BACK
     1      DISPOS       PERIF '/1H , 10X,'  EXC               MAT        SLAB
     2    CONC       FILL      BACK          U. D.'/1H ,36X,'CONC ',24X,'FIL
     3L ' )
      WRITE(5,38)EE,GM,BC,WC,BF,DE,PUD
   38 FORMAT(1H ,10X,' (CY)              (CY)       (CY)        (CY)       (CY)
     1       (CY)         (LF)' /1H0, 7X,F10.1,4X,6F10.1)
      GC TO  3
  200 CALL EXIT
      ENC
// DUP
*DELETE             PRIME
*STORE      WS  UA  PRIME
```

```
// JOB
// FOR
*EXTENDED PRECISION
*ONE WORD INTEGERS
*LIST ALL
*IOCS(CARD,DISK,1403 PRINTER)
      DIMENSION  A(30),B(13),DESC(38),C(51)
C     INPUT SECTION
C     READ STANDARD INFORMATION
      READ(2,2) A
    2 FORMAT(11F7.0)
C     READ VARIABLE DATA
    3 READ(2,4)(B(J),J = 1,11)
    4 FORMAT(11F7.0)
      READ(2,6) B(12),B(13),DESC
    6 FORMAT(2F7.0,38A1)
      IF (B(1))200,200,7
C     AER  TANK  VOL
    7 VOL  = ( (B(1) + B(3)* B(1))/7.48) * 60. * B(4)
C     ORGANIC LOAD
      C(2) = ( B(2) *  B(1) * 8.33 *1440. )/(1000000. *  VOL /1000.)
C     REQUIRED AER. VOL
      C(3) = (B(1) * 1440. * B(2) * 8.33 * 1000.)/(1000000.* B(5))
      IF(C(2) - B(5)) 8,8,10
    8 C(1) = VOL
      GO TO 12
   10 C(1) = C(3)
C     BOD TO MLSS RATIO
   12 C(4) =  (B(1)* 1440 * B(2) * 8.33) /(1000000. * B(6) * 8.33 *(
     1C(1) * 7.48 / 1000000.))
C     OXYGEN REQ
      C(5) = A(1) * (B(2) * B(1) * 1440 * 8.33 /(1000000. * 24.))
C      SOLIDS RETURN RATE
      C(6) = B(1) * B(3)
C     PERCENT BOD REMOVAL
      C(7) = (100. * A(20) * B(6) * A(2) * B(4)) /( 1 +(A(20) * B(6) * A
     1(2) * B(4)))
C     PERCENT BOD REMOVAL AT DESIGN TEMP
      EXP = A(21) - 20
      ASD =  A(20) * 1.05 **      EXP
      C(8) = (100. * ASD * B(6) * A(2) * B(4))/(1+ ASD * B(6) * A(2)
     1 * B(4) )
C     BIOLOGIC  SOLIDS  GEN
      C(9) = ((A(3)*B(1)*1440.*B(2)*8.33*C(7)/100.)/1000000.)-(A(14)*B(6
     1)*8.33*C(1)*7.48*A(2)/1000000.)
C     EXCESS SOLIDS VOL
      C(10) = C(9)          * 7.48 /(62.4 * A(4))
C     EXCESS SOLIDS PUMP RATE
      C(11) = C(10) /(1440 * A(29)/24)
C     THICKENER SURF A
      C(12) =    C(11) * 1440 / A(5)
C     FILTER AREA
      C(13) = C(9) /(A(6) * A(30))
C     SOLIDS OUTPUT
      C(14) =  C(9) * 100. /(62.4 * A(7) * 27 *100.)
C     SEDIMENTATION TANK SURFACE AREA
      C(15)  =  (B(1)  +  (B(3) * B(1))) * 1440./ B(7)
C     SED TANK  DIA
      C(16) = IFIX(((4*C(15)/3.1418)**.5)  +.5)
C     SED. TANK  VOL
      C(17)  =  C(15)*  ((A(8) + (A(8) +(C(16)* A(9)/24)))/2)
```

```
C      DETENTION TIME
       C(18)  =   C(17) /((B(1) + (B(3) * B(1)))* 60./7.48)
C      WEIR LOADING
       C(19)  =  B(1) * 1440. /(3.1418 * (C(16) -2))
C      BOD IN OUTFLOW
       C(20)  =  B(2)  * ( 1 - C(7)/100.)
C      AERATION HP
       C(21)  =  C(5) / 2.5
       CALL QUATY (C(16),A(15),B(11),A(17),A(16),A(8),A(13),C(22),C(23),
      1C(24),C(25),C(26),C(27),C(28))
C      AERATION TANK VOLUME
C      AVERAGE AER TANK AREA
       C(29)  = C(1) / B(8)
C      AVERAGE WIDTH
       C(30)  =  (C(29) / B(9)) **.5
C      AVERAGE LENGTH
       C(31)  =  B(9) * C(30)
C      BOTTOM LENGTH
       C(32)  = C(31)  - .5 * B(8) * A(10)*2.
C      BOTTOM WIDTH
       C(33)  = C(30)  - .5 * B(8) * A(10)*2.
C      LIQUID SURFACE LENGTH
       C(34)  =  C(31)  + .5 * B(8) * A(10)*2.
C      LIQUID SURFACE WIDTH
       C(35)  = C(30)  + .5 * B(8) * A(10)*2.
C      TOP LENGTH
       C(36)  = C(34) + 2 * A(12) * A(10)
C      TOP WIDTH
       C(37)  =  C(35)  + 2 * A(12) * A(10)
C      VOLUME EXCAVATION
       C(38) = ( C(32) * C(33) + ( (C(32) + 2* B(13)* A(10)) * (C(33)
      1 + 2 * B(13) * A(10)))) * (B(13) + A(18) + A(19)/12.) / 54
C      VCLUME EMBANKMENT
       CONST = B(8) + A(12) - B(13)
       C(39) = (      A(22)* CONST +((CONST*CCNST*A(10))/2.) + ((CONST*
      1CONST * A(11))/2.)) *  (2 *(C(36) +.5*A(22)) + 2 *(C(37) +.5 *
      2A(22))) / 27.
C      VOLUME OF SPOIL
       C(40) = C(38) - C(39)
C      VOL CONC
       XL = B(8) + A(12)
       RATS = XL * A(10)
       FACT = (XL**2 + RATS**2)**.5
       C(41) = (C(32) + 2 * FACT) *(C(33) + 2 * FACT) * A(19)/(27.*12.)
C      VOLUME OF MAT
       C(42) = (C(32) + 2 * FACT) * (C(33) + 2 * FACT) * A(18) /27.
C      CL2 TANK  VOLUME - CONTACT TANK
       C(43)  = B(1) * A(23) / 7.48
C      CL2 TANK AREA
       C(44)  = C(43) / A(24)
       WCL = ( C(44)/2)**.5
       TCL = 2*WCL
C      VCLUME EXCAVATICN  - CONTACT TANK
       AL = TCL + 2* A(27)/12. + 4.
       BL = WCL + 2* A(27)/12. + 4.
       C(45) = (( AL* BL) + (AL+ A(25)) * (BL + A(25))) / 2
       C(45)  = C(45) * (A(25) + A(26)/12. + A(17)) / 27.
C      VOLUME BASE CONC.
       C(46)  = (TCL + 2) * (WCL +2) * A(26) /(12.* 27.)
C      VCLUME OF WALL CONC
       C(47) = (2 *(TCL + 2*A(26)/12.)+ 2*WCL) * A(26) * (A(24) + A(28))/
      1324
C     VOLUME OF GRAVEL MAT
      C(48) = (TCL + 4)  * (WCL + 4) * A(17) / 27.
C     BACK FILL
      C(49)  = C(45) - C(46) - C(48) -((TCL + 2) * (WCL+2) * A(25))/27.
C      SPOIL
      C(50)  = C(45) - C (49)
C     CL2 REQUIREMENTS
      C(51)  = B(12) * B(1) * 1440 * 8.33 / 1000000.
C     OUTPUT SECTION - PRINT INPUT FIRST
      CALL HED1
      WRITE(5,25)DESC
   25 FORMAT(1H0,38A1,55X,'PAGE   1 '/,1H0,51X,'STANDARDS (INPUT)')
      WRITE(5,27)
   27 FORMAT(1H0,' OXYGEN    VOLATILE   SOLIDS   SS IN     SURFACE    FI
     1LTER    SOLIDS   S W D     BUTTOM SL SIDE SLP  SIDE SLP'/1H ,' UPT
     2AKE   SS IN MLSS GR CONST  SOLIDS     LOADING   LOADING   CONTENT
     3 SED TANK  SED TANK  AER. TANK AER TANK')
      WRITE(5,29)(A(K),K = 1,11)
   29 FORMAT(1H ,20X,'   A          OUTFLOW   TC THICK.   RATE   SOL. OUT
     1',20X,' INSIDE    OUTSIDE'/1H ,'(LB/LBBOD)  (PC)                 (PC
     2)    GAL/SF/DAY LBS/HR/SF  (PC)        (FT)       (IN/FT)'/1H0,     F7
     3.1,F10.3,F10.2,F10.3,2F10.1,F10.3,F10.1,3F10.1/)
      WRITE(5,31)
   31 FORMAT(1H0,' FREEBD   PREEBOARD  SOLICS   SED TANK  SED TANK  SED
     1TANK   AERATION  AERATION  RATE OF     MIN       WIDTH'/1H ,' AERATI
     2ON   SED TK.  GR CONST  SIDEWALL  BASE SLAB GRAVEL     TANK- GR  TK
     3 -CONC. BIO. REAC EXPECTED  TOP BERM')
      WRITE(5,33) (A(K),K=12,22)
   33 FORMAT(1H ,'  TANK                      B           THICK.    THICK.    THI
     1CK    MAT THICK THICKNESS    KS          TEMP'/1H ,'  (FT)        (FT)
     2               (IN)        (IN)       (FT)        (FT)        (IN)
     3       (DEG C)       (FT)'/1H0,F7.1,F10.1,F10.3,5F10.1,F10.6,2F10.2/)
      WRITE(5,35)
   35 FORMAT(1H0,'  CL2        CL2       EARTH AB  CL2 TK   CL2 TK   CL2 TK
     1    HRS- SOL. FILTER'/1H ,' CONTACT   TANK DEP. CL2 BASE  BASE SLA
     2B WALL TH   FREEBD    PUMPING   OPERATION'/1H ,'(MIN)       (FT)
     3    (FT)      (IN)       (IN)       (FT)        (HRS)      (HRS)')
      WRITE(5,37)(A(K),K = 23,30)
   37 FORMAT(1H0,F7.1,7F10.1,/////1H0,50X,'  VARIABLES(INPUT)')
      WRITE(5,39)
   39 FORMAT(1H0,'  INFLOW    INFLOW    RATE OF    AERATION ORGANIC    MIX
     1ED      HYD  LDG   WATER    LENGTH/     DESIGN   EARTH'/1H ,'
     2    (BOD)     SOLIDS     DETENTION LOADINGS  LIQ SS    SECONDARY  DEP
     3TH      WIDTH       TEMP     ABOVE')
      WRITE(5,40) (B(K),K = 1,11)
   40 FORMAT(1H ,22X,'RETURN',32X ,' CLARIFIER  AER TK     RATIO
     1        BASE'/1H ,'  (GPM)     (MG/L)       (PC)        (HRS)   (LBS BOD)
     2  (MG/L)    (GAL/SF/D)  (FT)                      (DEG C)    (FT)'/1H0,
     3F7.1,F10.1,F10.2,5F10.1,F10.2,2F10.1/)
      WRITE(5,42) B(12),B(13)
   42 FORMAT(1H0,'    CL2       EARTH'/1H ,' DOSAGE    ABOVE AER'/1H ,'
     1        TK BASE'/1H ,' (MG/L)       (FT)'/1H0,F7.1,F10.1)
      CALL HED1
      WRITE(5,44)DESC
   44 FORMAT(1H0,38A1,52X,'   PAGE     2'/1H0,57X,'OUTPUT')
      WRITE(5,46)
   46 FORMAT(1H0,'  AER       ORGAN     REQ AER   BOD/MLSS  OXYGEN      SOL
     1IDS     B O D     B O D     BIOLOGIC    EXCESS    EXCESS      THICK'/1H
     2 ,'  TK.VOL       LD         VOLUME       RATIO       REQ        RETURN     RE
     3MOVAL    REMOVAL     SOLIDS      SOL.     SOLIDS       SURF')
```

```
      WRITE(5,48)VOL,(C(K),K=2,12)
   48 FORMAT(1H,'(DET TIME)         (ORG LIM)                          ',
     *                'RATE      20 C     DESIGN T     GEN.       VOLUME
     1PUMP RATE  AREA'/1H ,'  (CF)    (LB/BOD1K)  (CF)                   (LB
     2/HR)   (GPM)      (PC)       (PC)       (LBS/DAY)    (GAL/D)  (GPM)
     3 (SF)'/1H0,F8.0,F9.1,F10.0,F10.2,8F10.1/)
      WRITE(5,50)
   50 FORMAT(1H0,'  FILTER   SOLIDS     SED TK     SED TK    SED TK     DET
     1       WEIR      B O D    AERATION  SED TK     SED TK    SED TK')
      WRITE(5,51)
   51 FORMAT(1H ,'  AREA     OUTPUT     SURF A      DIAM       VCLUME      TI
     1ME      LOADING   OUTFLOW     HP        EARTH EXC  GRAVEL     BASE')
      WRITE(5,52)(C(K),K=13,24)
   52 FORMAT(1H ,'   (SF)      (CY/D)     (SF)        (FT)        (CF)       (H
     1RS)   (GAL/LF/D) (MG/L)       (HP)         (CY)         (CY)        (CY)',
     21H0,F7.1,11F10.1/)
      WRITE(5,54)
   54 FORMAT(1H0,'SED TK     SED TK     DISP            PERIF       AER TK    AER
     1 TK     AER TK     AER TK      AER TK    AER TK      AER TK      AER TK')
      WRITE(5,55)
   55 FORMAT(1H ,'  WALLS     BACKFILL    EXCAV.    UN. DR.    AVG AREA  AVG
     1 W    AVG  L     BOTM L     BOTM W     LS    L     LS  W       TOP  L')
      WRITE(5,56)(C(K),K=25,36)
   56 FORMAT(1H ,'  (CY)        (CY)        (CY)        (LF)        (SF)        (F
     1T)        (FT)        (FT)        (FT)        (FT)        (FT)        (FT)'/1H
     20,F7.1,11F10.1/)
      WRITE(5,57)
   57 FORMAT(1H0,' AER TK     AER TK     AER TK     AER TK     AER TK     AER
     1 TK     CL2 TK     CL2 TANK  CL2 TK     CL2  TK     CL2 TK     CL2 TK')
      WRITE(5,58)
   58 FORMAT(1H ,' TOP W     EXC VOL    EMBK VOL   SPOIL VOL CONC VOL  GRA
     1VEL      VOL.         AREA     EXC VOL     BASE        WALL       GRAVEL')
      WRITE(5,59)(C(K),K=37,48)
   59 FORMAT(1H ,'  (FT)        (CY)        (CY)        (CY)        (CY)        (C
     1Y)        (CY)        (SF)        (CY)        (CY)        (CY)        (CY)'/1H
     20,F7.1,11F10.1/)
      WRITE(5,60)(C(K),K=49,51)
   60 FORMAT(1H0,' CL2 TK     CL2 TK     CL2 REQ'/1H ,' BACKFILL   SPOIL'/
     11H ,'  (CY)        (CY)       (LBS/DAY)'/1H0,F7.1,2F10.1)
      GO TO 3
  200 CALL EXIT
      END
// DUP
*DELETE             SECON
*STORE      WS  UA  SECON
```

```
// JOB
// FOR
*EXTENDED PRECISION
*LIST ALL
*ONE WORD INTEGERS
      SUBROUTINE QUATY (TK,SW,HT,GR,BO,   SP,FR,E,G,B,W,F,D,P)
      CW = SW/12
      TEMP = TK + (2*CW) + 4
      EE = (.785 * TEMP**2 + .785 * (TEMP + HT)**2)/2
      E = EE * (GR + BO/12. + HT)/27.
      G   =  ((3.1418/4.) * ((TK + 2*CW +4)**2)*GR) / 27.
      B   =  (3.1418/4.)  * ((TK + 2*CW +2)**2) * BO/(27.*12.)
      W   =   3.1418 *  (TK + CW) * CW  *  (SP + FR) /27.
      F   = E - G - B -(((3.1418/4.)* ((TK + 2*CW)**2) * HT)/27.)
      D  =  E - F
      P  =  3.1418 * (TK + 4)
      RETURN
      END
// DUP
*DELETE             QUATY
*STORE      WS  UA  QUATY
```

```
// JOB
// XEQ PRIME
  0.04   12.     1.      1.5    12.      7.    5.     0.15    1.     0.85    0.45
 570.  2000.   2500.   800.    24.      7.  AP FT FAIRFIELD PRIMARY SED COND A-1
 417.  2700.   3400.   800.    24.      7.  AP FT FAIRFIELD PRIMARY SED COND A-2
 278.  1600.   2100.   800.    24.      7.  AP FT FAIRFIELD PRIMARY SED COND A-3
```

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AP FT FAIRFIELD PRIMARY SED COND A-1

                                                STANDARDS (INPUT)

   SS        SIDE      BOTTOM     FREE-      SIDE      BOTTOM     FILTER    S S      GRAVEL     S S       B O D
 SLUDGE     WATER      SLOPE      BOARD      WALL       SLAB      LOADING  FILTER     BASE     REMOVAL   REMOVAL
 OUTFLOW    DEPTH                            THKNS      THKNS              OUTPUT    THKNS
  (PC)       (FT)     (IN/FT)     (FT)       (IN)       (IN)     (LBS/HR)  (P C)     (FT)       (PC)      (PC)

  0.04      12.00      1.00       1.50      12.00       7.00      5.00     0.15      1.00      0.85      0.45

                                          VARIABLE  DATA (INPUT)

                                 INFLOW     S S       B O D    SURFACE    FILTER     EARTH
                                           INFLOW    INFLOW    LOADING     OPS      HEIGHT
                                 (GPM)     (MG/L)    (MG/L) (GAL/SF/DAY) (HRS/DAY)  (FT)

                                 570.0     2000.0    2500.0    800.0      24.0       7.0


                                                    OUTPUT

  SURFACE    TANK      TANK        DETENT    WEIR        S S        SOLIDS    SLUDGE    FILTER     FILTER    B C D
   AREA      DIAM     VOL          TIME    LOADING      WEIR         TO      OUTFLOW    SOLIDS     AREA     WEIR
                                                         OUTFLOW    SLUDGE              OUTPUT              OUTFLOW
  (SQFT)     (FT)     (CF)         (HRS)  (GAL/LF/DAY)(MG/L)   (LBS/DAY)   (GPM)    (CY/DAY)    (SF)     (MG/L)

 1026.0      36.0   13081.5         2.8     7683.8      300.0    11623.3     24.1      41.3       96.8     1375.0


             EARTH         GRAVEL     BASE      WALL       BACK      DISPOS      PERIF
             EXC           MAT        SLAB      CONC       FILL      BACK        U. D.
                                      CONC                           FILL
             (CY)          (CY)       (CY)      (CY)       (CY)      (CY)        (LF)

             519.6          51.3      27.1      58.1      147.1      372.5       125.6
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                     PORTLAND - BANGOR - PRESQUE ISLE, MAINE

AP FT FAIRFIELD PRIMARY SED COND A-2
                                          STANDARDS (INPUT)
                                                                                      S S       B O D
   SS       SIDE     BOTTOM    FREE-     SIDE     BOTTOM    FILTER     S S    GRAVEL   REMOVAL   REMOVAL
 SLUDGE     WATER    SLOPE     BOARD     WALL      SLAB     LOADING  FILTER    BASE
 OUTFLOW    DEPTH                        THKNS     THKNS              OUTPUT   THKNS    (PC)      (PC)
  (PC)       (FT)    (IN/FT)   (FT)      (IN)      (IN)     (LBS/HR)  (P C)    (FT)

  0.04      12.00     1.00     1.50     12.00      7.00      5.00     0.15     1.00     0.85      0.45
                                        VARIABLE  DATA (INPUT)

                              INFLOW     S S      B O D    SURFACE     FILTER    EARTH
                                        INFLOW    INFLOW   LOADING      OPS      HEIGHT
                              (GPM)     (MG/L)    (MG/L) (GAL/SF/DAY) (HRS/DAY)  (FT)

                              417.0     2700.0    3400.0    800.0       24.0      7.0


                                              OUTPUT
                                    S S                                FILTER     FILTER     B O D
 SURFACE   TANK      TANK      DETENT   WEIR      WEIR       SOLIDS      SLUDGE    SOLIDS     AREA     WEIR
  AREA     DIAM      VOL       TIME     LOADING     OUTFLOW    TO        OUTFLOW   OUTPUT              OUTFLOW
                                                              SLUDGE
 (SQFT)     (FT)     (CF)      (HRS)  (GAL/LF/DAY)(MG/L)  (LBS/DAY)     (GPM)   (CY/DAY)   (SF)     (MG/L)

 750.6      31.0     9491.9     2.8     6590.5     405.0   11479.5       23.8      40.8      95.6    1870.0


         EARTH       GRAVEL    BASE      WALL      BACK     DISPOS     PERIF
         EXC         MAT       SLAB      CONC      FILL     BACK       U. D.
                                 CONC                       FILL
         (CY)        (CY)      (CY)      (CY)      (CY)      (CY)      (LF)

          412.3       39.8      20.7      50.2     130.0     282.3     109.9
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                               PORTLAND - BANGOR - PRESQUE ISLE, MAINE

AP FT FAIRFIELD PRIMARY SED COND A-3

                                                STANDARDS (INPUT)

   SS       SIDE      BOTTOM    FREE-     SIDE      BOTTOM    FILTER     S S      GRAVEL      S S       B O D
 SLUDGE     WATER     SLOPE     BOARD     WALL       SLAB     LOADING   FILTER     BASE      REMCVAL   REPOVAL
 OUTFLOW    DEPTH                         THKNS     THKNS               OUTPUT    THKNS
  (PC)      (FT)     (IN/FT)    (FT)      (IN)      (IN)     (LBS/HR)   (P C)     (FT)       (PC)      (PC)

  0.04      12.00     1.00      1.50      12.00     7.00      5.00      0.15      1.00       0.85      0.45

                                             VARIABLE  DATA (INPUT)

                               INFLOW     S S      B O D    SURFACE     FILTER     EARTH
                                         INFLOW    INFLOW   LOADING      OPS       HEIGHT
                               (GPM)     (MG/L)    (MG/L) (GAL/SF/DAY) (HRS/DAY)   (FT)

                               278.0     1600.0    2100.0    800.0       24.0       7.0



                                                    OUTPUT

 SURFACE   TANK     TANK       DETENT    WEIR        S S       SOLIDS    SLUDGE    FILTER     FILTER     B C D
  AREA     DIAM     VOL        TIME     LOADING     WEIR        TO      OUTFLOW    SOLIDS     AREA     WEIR
                                                    OUTFLOW   SLUDGE               OUTPUT               CUTFLOW
 (SQFT)    (FT)     (CF)       (HRS)  (GAL/LF/DAY)(MG/L)   (LBS/DAY)  (GPM)    (CY/DAY)    (SF)      (MG/L)

 500.4     25.0    6265.4       2.8    5539.8      240.0    4535.1      9.4       16.1       37.7     1155.0


       EARTH        GRAVEL    BASE      WALL      BACK     DISPOS     PERIF
       EXC          MAT       SLAB      CONC      FILL     BACK       U. D.
                              CONC                         FILL
       (CY)         (CY)      (CY)      (CY)      (CY)      (CY)      (LF)

       300.0         27.9     14.2      40.8     109.4     190.6      91.1
```

SECTION XIV

GLOSSARY, ABBREVATIONS AND SYMBOLS

activated carbon - Carbon particles usually obtained by carbonization of cellulosic material in the absence of air and possessing a high adsorptive capacity.

activated sludge - Sludge floc produced in raw or settled wastewater by the growth of zoogleal bacteria and other organisms in the presence of dissolved oxygen and accumulated in sufficient concentration by returning floc previously formed.

activated sludge process - A biological wastewater treatment process in which a mixture of wastewater and activated sludge is agitated and aerated. The activated sludge is subsequently separated from the treated wastewater (mixed liquor) by sedimentation and wasted or returned to the process as needed.

aerated pond - A natural or artificial wastewater treatment pond in which mechanical or diffused-air aeration is used to supplement the oxygen supply. See oxidation pond.

aeration tank - A tank in which sludge, wastewater, or other liquid is aerated.

aerobic bacteria - Bacteria that require free elemental oxygen for their growth.

alkalinity - The capacity of water to neutralize acids, a property imparted by the water's content of carbonates, bicarbonates, hydroxides, and occasionally borates, silicates, and phosphates. It is expressed in milligrams per liter of equivalent calcium carbonate.

ammonia - A chemical combination of hydrogen (H) and nitrogen (N) occurring extensively in nature. The combination used in water and wastewater engineering is expressed as NH_3.

anaerobic - Requiring, or not destroyed by, the absence of air or free (elemental) oxygen.

assimilative capacity - The capacity of a natural body of water to receive: (a) wastewaters, without deleterious effects; (b) toxic materials, without damage to aquatic life or humans who consume the water; (c) BOD, within prescribed dissolved oxygen limits.

biological process - (1) The process by which the life activities of bacteria and other microorganisms, in the search for food, break down complex organic materials into simple, more stable substances. Self-purification of polluted streams, sludge digestion, and all the so-called secondary wastewater treatments result from this process. (2) Process involving living organisms and their life activities. Also called biochemical process.

BOD - (1) Abbreviation for biochemical oxygen demand. The quantity of oxygen used in the biochemical oxidation of organic matter in a specified time, at a specified temperature, and under specified conditions. (2) A standard test used in assessing wastewater strength.

BOD load - The BOD content, usually expressed in pounds per unit of time, of wastewater passing into a waste treatment system or to a body of water.

centrifuge - A mechanical device in which centrifugal force is used to separate solids from liquids and/or to separate liquids of different densities.

chlorination - The application of chlorine to water or wastewater, generally for the purpose of disinfection, but frequently for accomplishing other biological or chemical results.

COD - Symbol for chemical oxygen demand.

coliform-group bacteria - A group of bacteria predominantly inhabiting the intestines of a man or animal, but also occasionally found elsewhere. It includes all aerobic and facultative anaerobic, Gram-negative, nonsporeforming bacilli that ferment lactose with production of gas. Also included are all bacteria that produce a dark, purplish-green colony with metallic sheen by the membrane-filter technique used for coliform identification. The two groups are not always identical, but they are generally of equal sanitary significance.

contact stabilization process - A modification of the activated sludge process in which raw wastewater is aerated with a high concentration of activated sludge for a short period, usually less than 60 min, to obtain BOD removal by absorption. The solids are subsequently removed by sedimentation and transferred to a stabilization tank where aeration is continued further to oxidize and condition them before their reintroduction to the raw wastewater flow.

detention time - The theoretical time required to displace the contents of a tank or unit at a given rate of discharge (volume divided by rate of discharge).

disinfection - The art of killing the larger portion of microorganisms in or on a substance with the probability that all pathogenic bacteria are killed by the agent used.

dissolved oxygen - The oxygen dissolved in water, wastewater, or other liquid, usually expressed in milligrams per liter, parts per million, or percent of saturation. Abbreviated DO.

dissolved solids - Theoretically, the anhydrous residues of the dissolved constituents in water. Actually, the term is defined by the method used in determination. In water and wastewater treatment the Standard Methods tests are used.

domestic wastewater - Wastewater derived principally from dwellings, business buildings, institutions, and the like. It may or may not contain groundwater, surface water, or storm water.

excess sludge - The sludge produced in an activated sludge treatment plant that is not needed to maintain process and is withdrawn from circulation.

extended aeration - A modification of the activated sludge process which provides for aerobic sludge digestion within the aeration system. The concept envisages the stabilization of organic matter under aerobic conditions and disposal of the end products into the air as gases and with the plant effluent as finely divided suspended matter and soluble matter.

fixed solids - The residue remaining after ignition of suspended or dissolved matter according to standard methods.

flotation - The process of removing suspended solids by induced air bubbles which cause the material to rise to the surface.

force main - A pressure pipe joining the pump discharge at a water or wastewater pumping station with a point of gravity flow.

industrial wastes - The liquid wastes from industrial processes, as distinct from domestic or sanitary wastes.

intercepting sewer - A sewer that receives dry-weather flow from a number of transverse sewers or outlets and frequently additional predetermined quantities of storm water (if from a combined system), and conducts such waters to a point for treatment or disposal.

inverted siphon - A pipeline crossing a depression or passing under a structure and having a reversal in grade on a portion of the line, thus creating a V- or U-shaped section of conduit. The line is under positive pressure from inlet to outlet and should not be confused with a siphon. Also called depressed sewer.

mechanical aeration - (1) The mixing, by mechanical means, of wastewater and activated sludge in the aeration tank of the activated sludge process to bring fresh surfaces of liquid into contact with the atmosphere.
(2) The introduction of atmospheric oxygen into a liquid by the mechanical action of paddle, paddle wheel, spray, or turbine mechanisms.

milligrams per liter - A unit of the concentration of water or wastewater constituent. It is 0.001 g of the constituent in 1,000 ml of water. It has replaced the unit formerly used commonly, parts per million, to which it is approximately equivalent, in reporting the results of water and wastewater analysis.

orthophosphate - An acid or salt containing phosphorus as PO_4.

outfall sewer - A sewer that receives wastewater from a collecting system or from a treatment plant and carries it to a point of final discharge.

oxidation pond - A basin need for retention of wastewater before final disposal, in which biological oxidation of organic material is effected by natural or artificially accelerated transfer of oxygen to the water from air.

pH - The reciprocal of the logarithm of the hydrogen-ion concentration. The concentration is the weight of hydrogen ions, in grams, per liter of solution. Neutral water for example, has a pH value of 7 and a hydrogen-ion concentration of 10^{-7}.

population equivalent - A means of expressing the strength of organic material in wastewater. Domestic wastewater consumes, on an average, 0.17 lb of oxygen per capita per day, as measured by the standard BOD test. This figure has been used to measure the strength of organic industrial waste in terms of an equivalent number of persons. For example, if an industry discharges 1,000 pounds of BOD per day, its waste is equivalent to the domestic wastewater from 6,000 persons ($1,000 \div 0.17 = 6,000$).

primary treatment - (1) The first major (sometimes the only) treatment in a wastewater treatment works, usually sedimentation. (2) The removal of a substantial amount of suspended matter but little or no colloidal and dissolved matter.

returned sludge - Settled activated sludge returned to mix with incoming raw or primary settled wastewater.

secondary wastewater treatment - The treatment of wastewater by biological methods after primary treatment by sedimentation.

sedimentation tank - A basin or tank in which water or wastewater containing settleable solids is retained to remove by gravity a part of the suspended matter. Also called sedimentation basin, settling basin, settling tank.

settleable solids - (1) That matter in wastewater which will not stay in suspension during a preselected settling period, such as one hour, but either settles to the bottom or floats to the top. (2) In the Imhoff cone test, the volume of matter that settles to the bottom of the cone in one hour.

sludge dewatering - The process of removing a part of the water in sludge by any method such as draining, evaporation, pressing, vacuum filtration, centrifuging, exhausting, passing between rollers, acid flotation, or dissolved-air flotation with or without heat. Involves reducing from a liquid to a spadable condition rather than merely changing the density of the liquid (concentration) on the one hand or drying (as in a kiln) on the other.

suspended solids SS - (1) Solids that either float on the surface of, or are in suspension in, water, wastewater, or other liquids, and which are largely removable by laboratory filtering. (2) the quantity of material removed from wastewater in a laboratory test, as prescribed in "Standard Methods for the Examination of Water and Wastewater" and referred to as nonfilterable residue.

total solids TS - The sum of dissolved and undissolved constituents in water or wastewater, usually stated im milligrams per liter.

vacuum filter - A filter consisting of a cylindrical drum mounted on a horizontal axis, covered with a filter cloth, and revolving with a partial submergence in liquid. A vacuum is maintained under the cloth for the larger part of a revolution to extract moisture. The cake is scraped off continously.

volatile solids - The quantity of solids in water, wastewater, or other liquids, lost in ignition of the dry solids at 600°C.

SECTION XV

APPENDIX

PRELIMINARY SUBSURFACE INVESTIGATION PROPOSED POLLUTION CONTROL FACILITIES THE NORTHERN MAINE REGIONAL TREATMENT SYSTEM

INTRODUCTION

The purpose of this study has been to make a preliminary investigation of the subsurface conditions in the vicinity of proposed pollution control facilities proposed in the Research and Development phase of the Northern Maine Regional Treatment System. It is understood that the proposed facilities are to consist of works for the conveyance and treatment of domestic and industrial sanitary sewage from Washburn, Presque Isle, Easton Station, Caribou, and Grimes Mill.

The investigation has been accomplished through a review of available airphotos and soil and geologic maps, a geologic field reconnaissance, field explorations, and laboratory testing and analysis. The airphotos and maps reviewed were: (1) Aerial photographs of Project Area, Proj 16110 DPT, by James W. Sewall Company of Old Town, Maine, dated May 15, 1971; (2) Soil Survey, Aroostook County, Maine, Northern Part, Series 1958, No 27, USDA Soil Conservation Service, issued April, 1964; and (3) Preliminary Geologic Map of Maine, Maine Geological Survey, 1967. The geologic field reconnaissance was made in mid-October, 1971, by Mr. S. E. Walker of Jordan Gorrill Associates.

The subsurface conditions were explored by means of 43 test borings made by Maine Test Broings, Inc, of Brewer, Maine, in November and December, 1971. The locations of the borings (except the three made at Washburn which are not shown), as selected by Jordan Gorrill Assoicates, are indicated on the Boring Location Maps, Figures 67 and 68. Logs of the borings based on the drillers' field notes and modified on the basis of laboratory examination and evaluation of the samples and field data are presented as data sheets 3 through 45. A key to symbols and terms used on the logs is included as data sheet 2.

All samples retrieved from the borings were visually examined in the laboratory. Representative samples were selected for grain size analysis, the results of which are plotted on data sheets 46, 47 and 48. A qualitative evaluation of the moisture condition of each of the samples based on visual examination as received at the laboratory was made and is recorded at the appropriate location on the logs.

DESCRIPTION AND EVALUATION OF SITE CONDITIONS

GENERAL - As anticipated from airphoto and map reviews, the soils in the study area were found through the field reconnaissance and explorations to be a variety of glacial (late Wisconsin Age) and post-glacial ice-,

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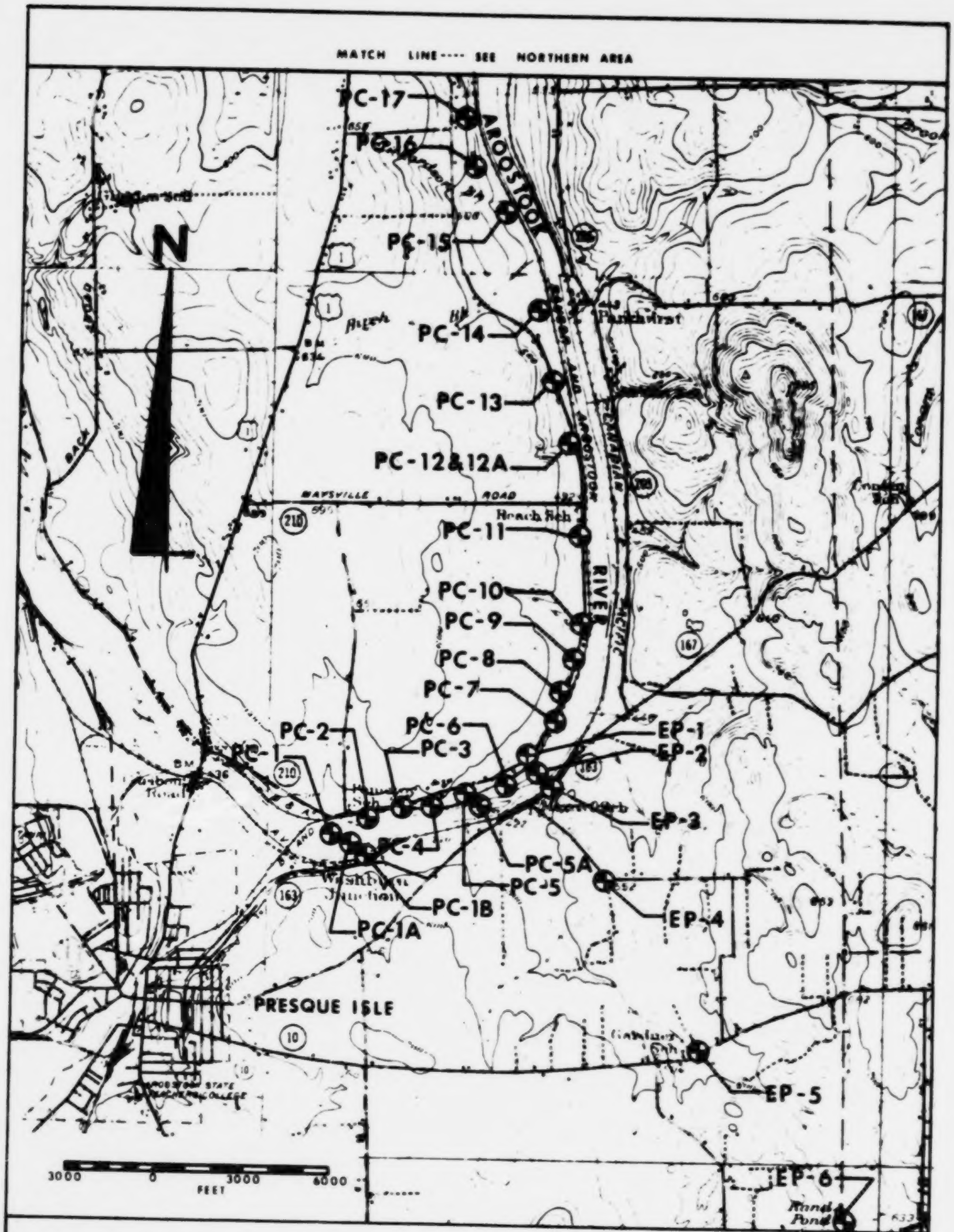
MATCH LINE ---- SEE NORTHERN AREA
N
PC-17
PC-16
PC-15
PC-14
PC-13
PC-12&12A
PC-11
PC-10
PC-9
PC-8
PC-7
PC-6
PC-5A
PC-5
PC-4
PC-3
PC-2
PC-1
PC-1A
PC-1B
EP-1
EP-2
EP-3
EP-4
EP-5
EP-6
AROOSTOOK
RIVER
MAYSVILLE ROAD
PRESQUE ISLE
3000
0
3000
6000
FEET

FIGURE 67
CORE AREA BORING LOCATIONS
SOUTHERN AREA

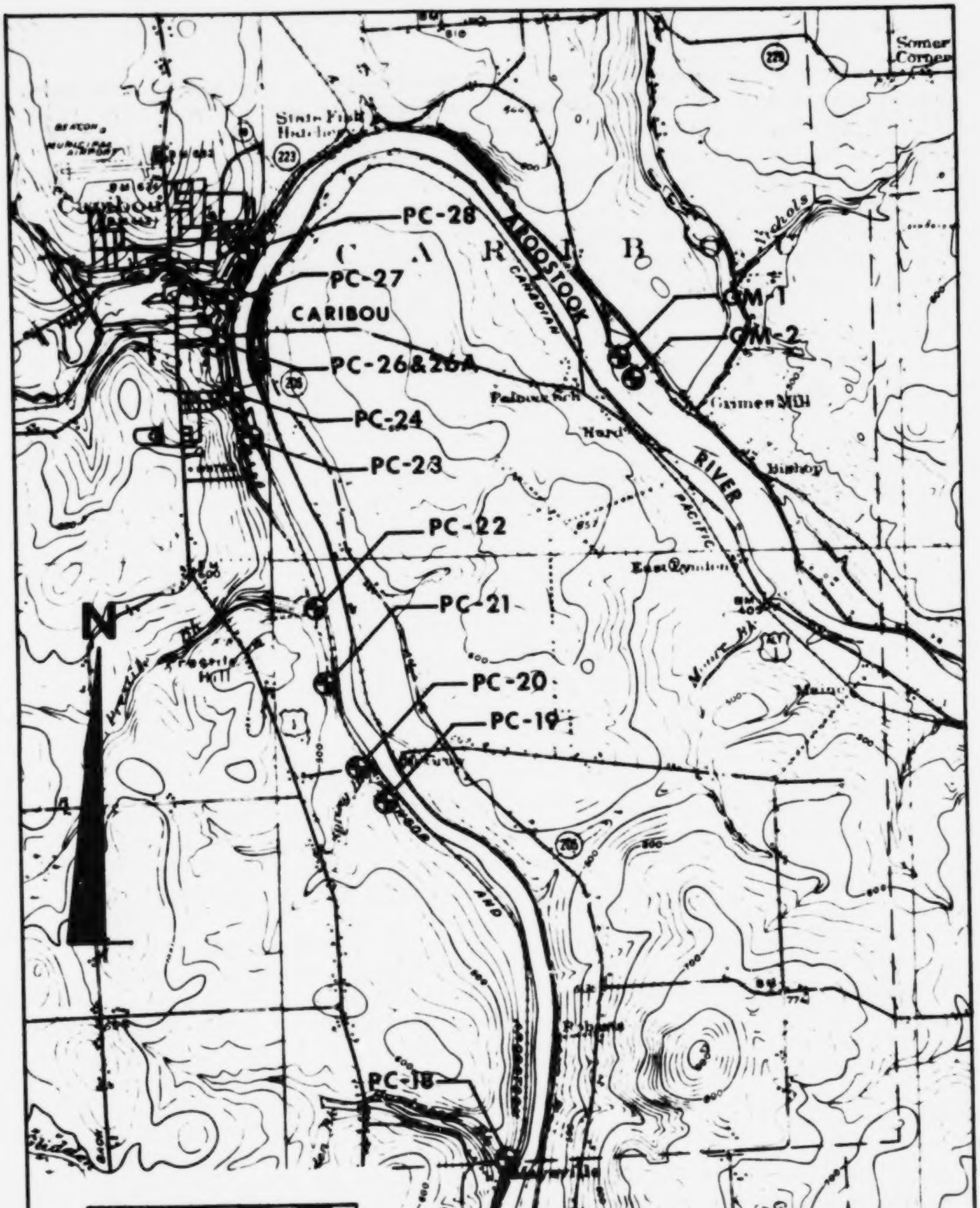
PC-28
C A R I B
AROOSTOOK
PC-27
CARIBOU
PC-26&26A
PC-24
PC-23
PC-22
PC-21
PC-20
PC-19
PC-18
GM-1
GM-2
RIVER
PACIFIC
State Fish Hatchery
223
205
Grimes Mill
Bishop
Somer Corner
N
3000 0 3000 6000
FEET
MATCH LINE---- SEE SOUTHERN AREA

FIGURE 68
CORE AREA BORING LOCATIONS
NORTHERN AREA

water-, and wind deposited soils over the undulating surface of the dark grey metamorphosed limestone common throughout the area. Generally, the soils of the rolling upland areas consist of only a few feet of firm to compact till (an unstratified, well-graded mixture of clay to gravel and boulder sized particles) over the sometimes weathered bedrock surface, while the stream and river valleys are underlain by some tills and a variety of loose to compact stratified, water-laid deposits of silts, sands, and gravels over the erratically shallow (exposed at ground surface) to deep (greater than 31.5 at Caribou Stream) bedrock surface. Groundwater levels appear to vary with the local terrain and soil conditions, and are expected to fluctuate considerably (probably as much as 5 to 8 feet) with the seasons and climatic conditions. Free groundwater was not encountered in many of the shallow (less than 12 feet deep) borings.

Brief descriptions and evaluations of the site conditions at the several sections of the study area are presented in the following paragraphs.

WASHBURN TREATMENT PLANT SITE - The proposed Washburn treatment plant site (upland site) is located on a broad terrace above the floodplain of Salmon Brook about a quarter of a mile north of Wade Road and west of the Brook. The subsurface conditions were explored by Borings W-1, -2, and -3. Logs of these borings are presented as data sheets 3, 4 and 5, and again size distribution curves for four representative samples are plotted on data sheet 46. As may be seen from the logs, the site is underlain by 8 to 9 feet of firm to compact sandy gravel which is in turn underlain by about 8 to 13 feet of firm silty sand and firm sandy siltt extending at least to the exploration depths of 26.5 feet. Based on the moisture condition of the samples, it appears that the groundwater table was at or below a depth of 20 feet in late November, 1971. The soils have high to moderate bearing strengths and would be expected to provide adequate support for the proposed basin, tank, and control facilities.

GRIMES MILL TREATMENT PLANT SITE - The proposed Grimes Mill treatment plant site is located on a broad terrace, about 25 to 30 feet above the Aroostook River, west of State Route 223 and about a half mile north of the confluence of the Little Madawaska and Aroostook Rivers. The subsurface conditions were explored by Borings GM-1 and -2, located approximately as shown on the Subsurface Exploration Map, Figure 68. Logs of these borings are presented as data sheets 6 and 7, and grain size distribution curves for two representative samples are plotted on data sheet 47. The borings encountered firm to compact water-laid deposits of sand and gravel, with some thin clay and silt layers below 25 feet in GM-1, extending to the bottoms of the borings at a depth of 26.5 feet. Although some of the near-surface samples were found to be damp, the groundwater table was apparently at least 25 feet below the ground surface in mid-December, 1971. The soils have moderate to high bearing strengths and would be expected to provide adequate support for normal treatment facilities.

POTATO SERVICE, INC TREATMENT PLANT SITE - The existing Potato Service treatment facilities are located on the northerly bank about 15 to 20 feet above the Aroostook River adjacent to the company's potato processing plant approximately one mile east of US Route 1 on State Route 210. The subsurface conditions were explored by borings PC-1 (near the processing plant), PC-1A and -1B (through the dikes of the aeration basins of the existing treatment facilities), and PC-2 (about 100 feet north of the existing basins). The approximate locations of these borings are shown on Figure 67. Logs of these borings are presented as data sheets 8 through 11. As may be seen from the logs, the borings encountered 0 to 14 feet of loose to firm gravelly silty sand fill over firm stratified silty sand at 8 feet in PC-1 and over firm to compact and stiff glacial till in the other three borings. The refusal to sampler penetration at 26.0 feet in PC-1A may have been caused by a boulder, but the refusal encountered at 8.0 feet in PC-2 is believed to have been on the bedrock surface. The natural groundwater table appears to have been at least 8 feet below the surface in late November, 1971. The undisturbed native soils would be expected to provide suitable support for additional structures or earthfill embankments. However, the loose surficial fills and probably shallow bedrock would be expected to cause some difficulty in making any deep trench excavations.

PRESQUE ISLE TO CARIBOU INTERCONNECT - The proposed 6 to 8 foot conduit from Presque Isle to Caribou is to be located along the westerly bank of the Aroostook River and extend from the vicinity of the existing treatment facilities at the Potato Service Inc plant to the vicinity of the existing municipal treatment plant in Caribou. The conduit invert is to slope from elev 438± (USGS) at Sta 0+000 (preliminary project stationing) at the Potato Service Inc plant to elev 425± at Sta 17+500, drop to elev 423± at Sta 17+500, slope to elev 403± at Caribou Stream, Sta 67+500±, and then slope into the municipal treatment plant at Sta 71000± (ground surface at elev 390±). The proposed sewage pipeline from Easton Station is to discharge into this conduit at Sta 7+600±.

The subsurface conditions along the proposed route were explored by 31 borings numbered PC-1 to -28 (with auxiliary numbers) and EP-1. The approximate locations of these borings are indicated on Figures 67 and 68. The logs are presented as data sheets 8 and 11 through 40. Grain size distribution curves for nine representative samples are plotted on data sheet 48.

As may be seen from the logs, the borings encountered a variety of soil, rock, and groundwater conditions. Generally, the soils along the proposed route were found to consist of various combinations of glacial till, stratified river terrace (including glacial outwash and recent alluvium) deposits, and miscellaneous man-placed fills overlying the irregular bedrock surface. Groundwater levels vary with both terrain and season from depths of less than 1 foot to greater than 15 feet below the ground surface.

With reference to the preliminary project stationing and proposed conduit invert elevations, we would expect the following subsurface conditions:

0+000 to 1+500	0 to 8 feet of fill over 5 to 10 feet of till. Bedrock below invert.
1+500 to 7+000	0 to 10 feet of till over bedrock, 0 to 20 feet above invert. Some groundwater near bedrock surface.
7+000 to 10+000	10+ feet of till. Bedrock below invert.
10+000 to 14+000	0 to 10+ feet of fill and river terrace deposits over till. Bedrock below invert. Groundwater at or below invert.
14+000 to 15+750	4 to 10 feet of terrace deposits and till over bedrock, 0 to 4 feet above invert. Groundwater at or below invert.
15+750 to 19+500	4 to 10 feet of terrace deposits and fill over till. Bedrock below invert. Groundwater near top of till, 0 to 3 feet above invert.
19+500 to 21+150	10 to 15 feet of terrace deposits over till. Bedrock below invert. Some groundwater near top of till, 2 to 4 feet above invert.
21+150 to 30+500	0 to 6 feet of terrace or outwash deposits over till, 0 to 12 feet above invert. Bedrock below invert. Some groundwater near top of till, below to 3 feet above invert.
30+500 to 37+000	0 to 15 feet of terrace, outwash, and alluvial deposits over till. Till and bedrock below invert. Groundwater below invert.
37+000 to 50+000	0 to 5 feet of terrace or outwash deposits, over 4 to 10 feet of till, over bedrock at invert +2 to -2 feet. Groundwater near surface of till, 0 to 5 feet above invert.
50+000 to 52+500	0 to 3 feet of terrace deposits, over 10 feet of till. Bedrock below invert. Groundwater at or below invert.

52+500 to 55+000	10 to 25 feet of till over bedrock, 0 to 6 feet above invert. Groundwater in till 5 to 10 feet above invert.
55+000 to 56+000	Same as above except bedrock below invert.
56+000 to 63+500	5 to 30+ feet of terrace or outwash deposits over till. Bedrock below invert. Groundwater below invert.
63+500 to 61+000+	0 to 6 feet of fill or terrace and outwash deposits, above invert. 0 to 30 feet of outwash over till, below invert. Bedrock below invert. Groundwater below invert.

Although much of the required excavation work is not expected to encounter any unusual construction difficulties, some sections will probably require one or more of the following: rock excavation, sheeting and bracing, and control of surface and groundwater seepage. The areas of deep cut will need considerably more exploration and evaluation prior to final design and construction. Control of seepage in excavations made entirely in nearly impervious tills is not expected to be difficult, probably requiring only sump pumping. However, seepage control in excavations below the groundwater table in the pervious terrace, outwash and alluvial deposits will warrant careful consideration and may require some braced close sheeting and sump pumping and possibly some wellpoint dewatering.

The conduit will require an elevated structure for the Caribou Stream crossing. The results of Boring PC-27 indicate that soils below a depth of about 9 feet should have sufficient bearing strength to support such a structure.

EASTON STATION TO PRESQUE ISLE - The proposed 14- to 24-inch pipeline from Easton Station to the proposed Presque Isle to Caribou conduit is to extend from the industrial complex at Easton Station, along the Bangor & Aroostook Railroad tracks to Rand Pond, down, northwesterly, to and across the Aroostook River approximately three-quarters of a mile southwest of the intersection of State Routes 163 and 205. The subsurface conditions were explored by Borings EP-1 (at the proposed Presque Isle to Caribou conduit), EP-2 and -3 (on the westerly and easterly banks of the river), and EP-4, -5 and -6 (along the proposed pipeline from Easton Station). The approximate locations of these borings are indicated on the Subsurface Exploration Map, Figure 67. The logs are presented as data sheets 40 through 45 and grain-size distribution curves for two representative samples are plotted on data sheet 47.

Boring EP-1 encountered firm to compact sandy silt till extending to the completion depth of 11.5 feet. Although the samples were damp, no free groundwater was observed during the drilling of the hole. Excavations, within the explored depth, would not be expected to pose any unusual difficulties.

Boring EP-2, on the westerly bank (ground surface about 12 to 15 feet above the river level), encountered 4 feet of loose (probably flood-deposited) silty fine sand over 8 feet of compact to very compact sandy silt till which was in turn underlain by a water-laid deposit of very compact gravelly sand extending to the 26.5-foot completion depth of the boring. The field and laboratory observations indicate the groundwater table was between 12 and 15 feet below the surface (about at the river level) in early December, 1971. Boring EP-3, near the easterly bank (ground surface about 30 feet above river level), encountered several firm to compact strata of water-deposited silts, sands, and gravels before being terminated by refusal (probably on bedrock, but possibly on a boulder) at the depth of 15.3 feet. the borehole and samples were dry to the depth explored. On the two banks, excavations extending to elevations above river level are not expected to encounter any unusual difficulties except possibly some rock excavation in the vicinity of EP-3 depending upon the required excavation depth. However, excavations extending into the clean granular deposits below river level will probably require close sheeting and bracing and possibly wellpoint dewatering to maintain trench stability. Based on the limited information and observations currently available, little or no rock excavation is expected for the river crossing itself.

The soils along the cross-country pipeline from the crossing of State Route 163 to Easton Station are expected to consist of firm to compact shaley glacial tills from 0 to more than 10 feet thick over the undulating and sometimes weathered limestone-shale bedrock surface. Groundwater levels vary with both season and terrain from depths of less than 1 foot to greater than 10 feet, sometimes being well below the bedrock surface. With regard to the preliminary stationing (in 1000-foot stations) which has been assigned, we would expect the depths to bedrock to vary approximately as follows:

Stations	Depth to Bedrock
0+250 (edge of river) to 0+750	6 to 15+ feet
0+750 to 1+500	2 to 6
1+500 to 3+000	0 to 6
3+000 to 4+500	6 to 10+
4+500 to 6+000	0 to 6
6+000 to 10+000	3 to 6
10+000 to 12+000	6 to 10+
12+000 to 14+000	0 to 6
14+000 to 19+000 (B&A RR tracks)	3 to 6
19+000 to Easton Station	4+ to 6+

The necessity for a considerable amount of rock excavations is expected to be the major construction difficulty; however, control of surface and groundwater, especially in any deep cuts and in the low-lying areas along the stream will also warrant careful consideration.

LIMITATIONS

The descriptions and evaluations of the site conditions as presented herein are based on an interpretation of a limited number of widely spaced explorations, on-site observations of surficial features, and a review of available mapping and photographic information. The evaluations are preliminary in nature and scope and are intended for use in preliminary engineering planning and cost estimating only. Additional detailed exploration, testing, and analysis will be needed to establish appropriate design criteria for the several proposed facilities.

DATA SHEET 2
NOTES

1.	w	Water content, percent - dry weight basis.
2.	c	Unit cohesion, tons/sq ft - based on Unconfined Compression testing.
3.	q_p	Unconfined compressive strength - by pocket penetrometer.
4.	c_v	Unit cohesion, tons/sq ft - based on Field Vane Shear Testing.
5.	C____	Groundwater surface (casing in place) - based on observation by the boring crew.
6.	____	Groundwater surface (casing removed) - based on observation by the boring crew.
7.	////	Hole caved in and dry at depth indicated.
8.	LL	Liquid limit.
9.	PI	Plasticity index.
10.	WOH	Weight of hammer.
11.	WOR	Weight of rods.

SECTION XVI

SUMMARY REPORT

As part of its program for pollution abatement, the NMRPC has actively pursued public information programs to inform the citizens and industries of its activities. Because of the length and technical nature of both the "Research and Development" and the "River Basin Planning" reports, the NMRPC felt that it would be difficult for the public to read and comprehend them. The NMRPC decided, therefore, to prepare a summary report to inform the general public on the nature of the studies undertaken.

The summary report contains discussions on the historic background of the project, the nature of the problem, the effects of existing waste loads on the river, techniques of waste treatment, the pollution control alternatives, the research and development efforts, cost analysis, the land use and population studies, the economy of the area, legal and administrative considerations for implementation, and the basic conclusions and recommendations of the project. In order to make the summary report as effective as possible, graphics were employed to illustrate basic treatment processes and to define the geographic limits of study area (center fold of report). In addition, photographs were employed to illustrate the nature of specific problems, and the general characteristics of the area.

A draft of the summary report was prepared by the Edward C. Jordan Co., Inc. and edited by the NMRPC. To provide maximum impact on the reader, it was decided that the cover and center fold map of the area should be multicolored. Upon completion of the draft, a professional printing firm was contracted with to print 5,000 copies of the summary report. The initial distribution of the report was made at a public hearing held in Presque Isle, Maine, on April 26, 1972. Since that date, the summary report has been distributed by the NMRPC to numerous civic organizations, and local, state, national, and international government officials. A representative distribution list is as follows:

Chamber of Commerces
Rotary Clubs
St John River Basin Board
Local Community Officials, including:
    Town Selectmen, Town Council Members
    Planning Boards, Sewer Districts, and
    Water Districts
Nato-CCMS-Inland Watter Pollution Project, United States
    Symposium, Fish River Lake, Maine, Sept 18-23, 1972

Interested parties can obtain copies of the summary report upon request from the NMRPC.

☆U.S. GOVERNMENT PRINTING OFFICE 1973 514-155/309 1-3